CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

SEPTEMBER 1961 **VOL. LVI. NO. 9**

FIFTY-SIXTH YEAR OF PUBLICATION

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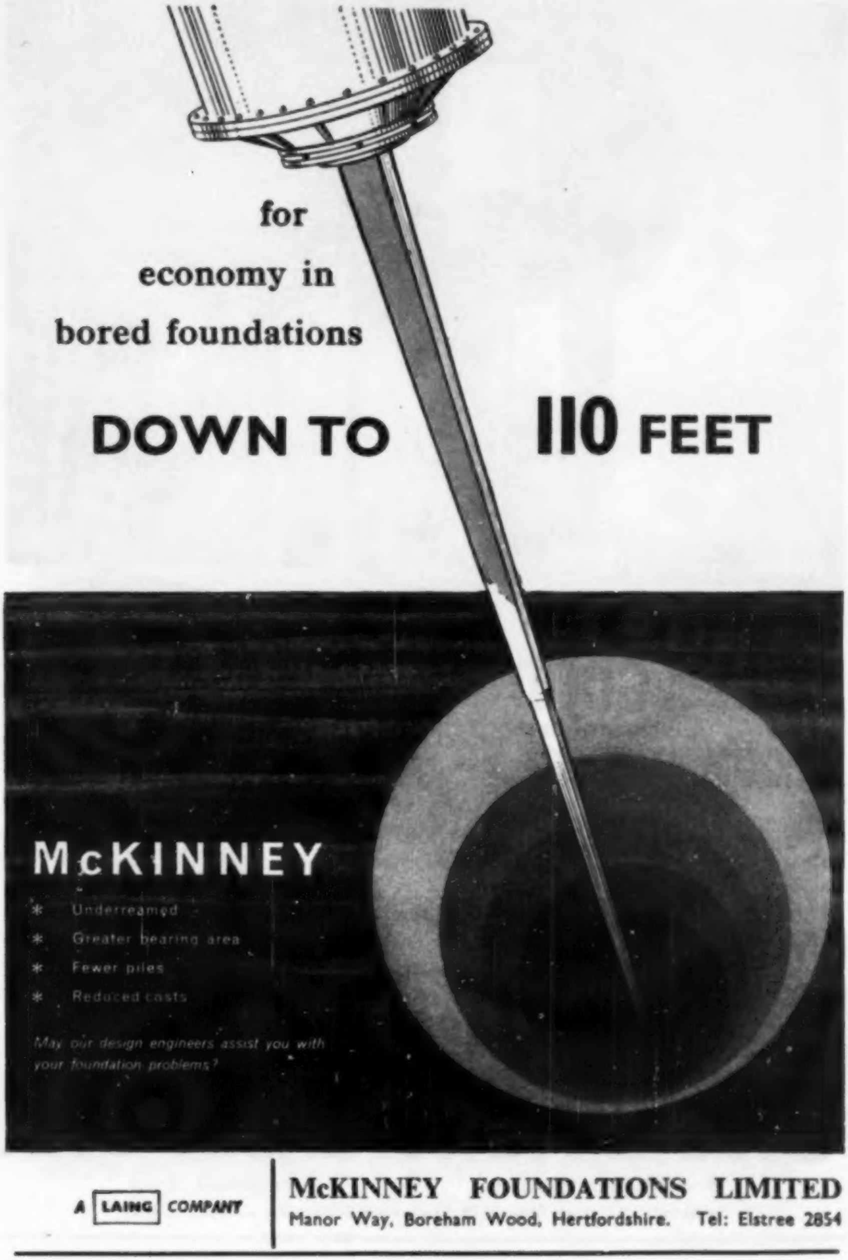
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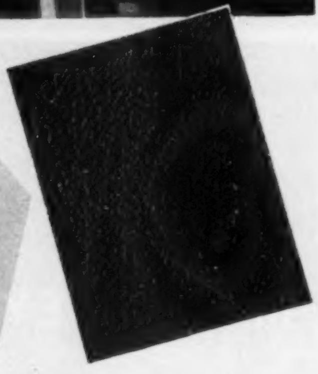

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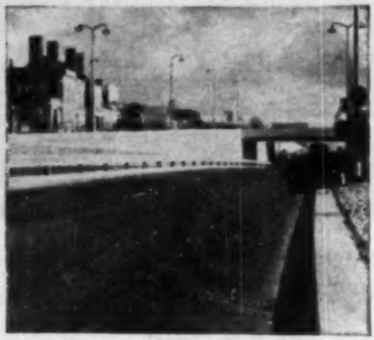

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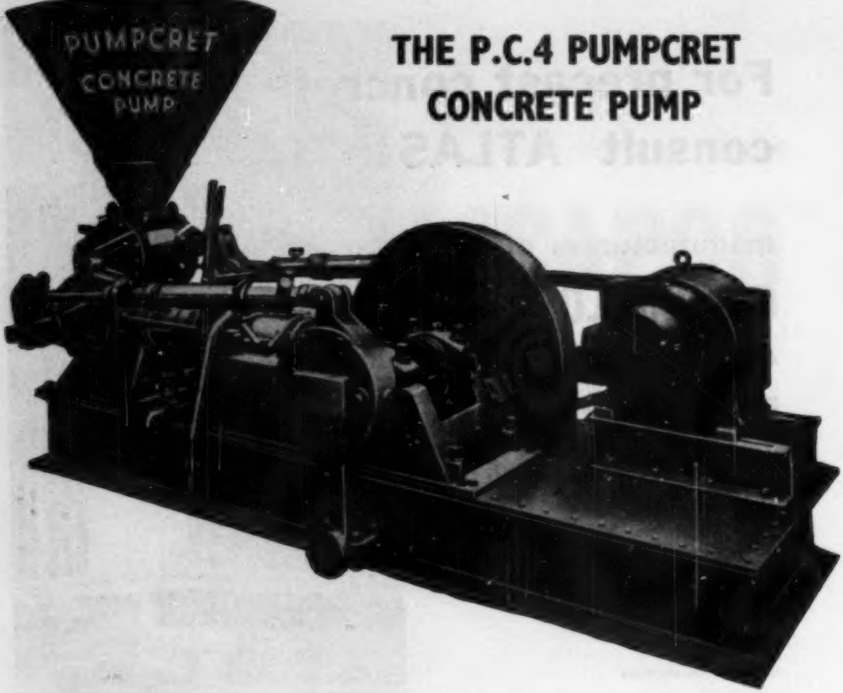

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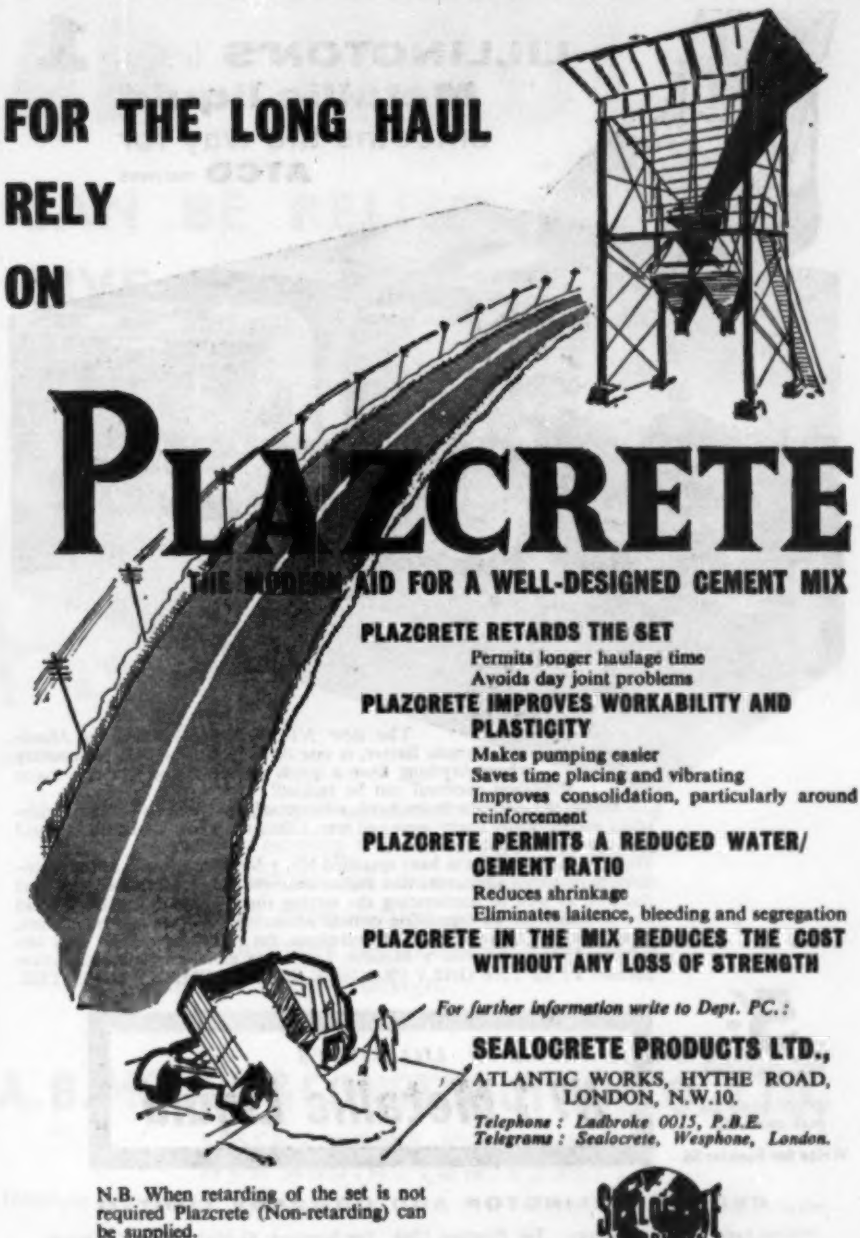
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LVI, No. 9. LONDON, SEPTEMBER, 1961.

EDITORIAL NOTES

Earthquake-resistant Construction.

DURING the past two or three decades there has been considerable development in the theory and practice of designing structures to resist the effects of earthquakes. At one time, the simplicity of the method was such that a building was designed to resist a horizontal force equal in magnitude to, say, one tenth its own weight or of the dead weight of that part of the structure above any horizontal plane being considered. The complexity of the subject today is seen from the contents of the papers presented at the Second World Conference on Earthquake Engineering, which was held last year in Tokyo and elsewhere in Japan. The proceedings * of the Conference were published recently in the form of three volumes containing upwards of two thousand pages and a supplementary volume giving without comment the regulations applying to earthquake-resistant construction in about a dozen countries.

Some regulations, such as those of New Zealand, Portugal, Italy and Austria, are comparatively brief and specify a simple coefficient to be applied to the weight of the structure. Others such as those of France, California and Turkey are more elaborate in the assessment of the horizontal force; some countries including Canada and Venezuela adopt the U.S.A. formula. Other regulations, the Greek for instance, give a wealth of structural details. Probably the most complex and comprehensive are the Japanese regulations as might be expected in view of the fact that Japan has suffered much from severe earthquakes. The numerous factors which it is necessary to take into account include the weight and height of the structure, part of the live load, the probability of a snow load, the nature of the soil, the locality, and the material and nature of the construction; basic coefficients up to 0·3 or greater (compared with the common factor of 0·1) are applicable to some structures in certain districts.

The principal subjects dealt with in the hundred and twenty-four papers

* "Proceedings of the Second World Conference on Earthquake Engineering." Three volumes (and a supplementary volume). Price 17.00 U.S.A. dollars. Obtainable from Association for Science Documents Information, Institute of Technology, Tokyo, Japan.

presented at the Conference include soils and foundations, structural response, ground movements, earthquake-resistant design, and recent earthquakes. One of the primary conclusions is that theoretical study of the effects of earthquakes is valueless unless well supported by practical data. Examination of damage immediately after an occurrence is more rewarding than theoretical and experimental investigations. Earth movements due to seismic effects in the U.S.A. have been recorded in great detail and the records have been used effectively in research connected with earthquake-resistant structures. The influence of the type of ground on these effects is now well established but research dealing with the vibrational characteristics of soils continues, and the opinion was expressed at the Conference that this aspect of the problem deserves attention equal to that given to the more popular problem of structural response. In Japan, instruments have been installed in various parts of many buildings and some correlation between the behaviour of buildings and the type of soil is being obtained. During a severe earthquake, the material in many structures is likely to be stressed beyond the limit of elasticity. Some structures resist severe seismic effects because of plastic deformation. Therefore many papers dealt with the elasto-plastic response of structures and it was concluded that the development of a method of design based on the limitation of deformation is foreseeable. Another method of design that is claimed to be new is based on energy absorption.

Accounts of many recent earthquakes are given in the Proceedings and the effects on buildings, dams and other structures are described in sufficient detail to be of use in determining means of resisting seismic effects. The earthquake at Agadir in 1960, although disastrous, was not among the most violent shocks on record and, from a technical point of view as well as on humane grounds, this circumstance was fortunate because it allowed the most resistant buildings to survive and others less resistant to suffer damage roughly inversely to their resistance, thereby enabling the strong and weak points of modern construction to be examined.

New developments produce new problems. Damage of a nuclear reactor by earthquakes could lead to uncontrolled nuclear fission with the possibility of explosion and release of fission products, with consequent hazard to human life. Therefore these structures, such as that currently being constructed in Japan, are designed for the most severe seismic conditions and the detailed analysis must be more thorough than for structures of other types. It is worth while to draw attention to a method of building construction in Japan that has a high degree of resistance to seismic effects. In this method a framework of light lattice steelwork is erected first and is encased in reinforced concrete (not merely concrete with a minimum of encasement reinforcement). The resistance of such composite construction is considered to be the sum of the separate resistances of the steelwork and the reinforced concrete.

Although designers of structures to be erected in this country do not take seismic effects into account, the British Isles are not entirely free from natural seismic phenomena, although these are by no means so disastrous as those occurring in parts of the world well known for the severity of earthquakes. There are actually over a thousand authoritative records of earth tremors in Great Britain and many more of dubious authenticity. The vast majority of these occurrences were slight but some were sufficiently severe to cause structural

damage, and, in a few instances, fatalities. The year 1750 was known as the "earthquake year", and since then tremors, sufficiently severe to cause cracks in buildings and chimneys to fall from house-tops, have been centred at Inverness (in 1769, 1816 and 1901), Derbyshire (in 1795), Comrie, Scotland (in 1839 and 1841), Hereford (in 1863 and 1896) and Swansea (1906). Few parts of the country, including London, have been entirely free of slight tremors at some time during this period. The most severe earthquake in this country was in 1884 when more than a thousand buildings in and around Colchester suffered damage, a peculiarity being that the tremors occurred in a district that had hitherto been free from disturbances. Most other occurrences are associated with geological faults and it would appear that, even in this seemingly tranquil land, seismic disturbances in the vicinity of such faults are not unlikely. Ground shocks arising from intentional or accidental explosions may resemble natural seismic effects and to this extent a knowledge of earthquake-resistant construction is useful, but such knowledge is imperative if designers are concerned, as nowadays so many in this country are, with structures that are to be built in parts of the world where earthquakes of various degrees of severity are not only likely occurrences but are highly probable.

Responsibility for Constructional Costs.

WITH reference to the Editorial Note on "Structural Safety" in this journal for July last, and in particular with reference to the responsibility of constructors under French law, a reader sends the following extract concerning the financial obligations of architects, a term inclusive of structural engineers in the far-off days to which the extract applies.

"In the renowned and spacious Greek city of Ephesus, a law is said to have been made of old by the forefathers of the citizens, in harsh terms but not unjust. For when an architect undertakes the erection of a public work, he estimates at what cost it will be done. The estimate is furnished, and his property is assigned to the magistrate until the work is finished. On completion, when the cost answers to the contract, he is rewarded by a decree in his honour. If not more than a fourth part has to be added to the estimate, the state pays it and the architect is not mulcted. But if more than a fourth extra is spent in carrying out the work, the additional sum is exacted from the architect's property.

Would that the Gods had impelled the Roman people to make such a law not only for public, but also for private buildings! In that case unqualified persons would not swagger abroad with impunity, but persons trained in entirely accurate methods would profess architecture with confidence. Nor would owners be led on to unlimited and lavish expenditure, so that they are even dispossessed of their property; and the architects themselves, controlled by the fear of a penalty, would be more careful in calculating and declaring the amount of the cost. In this way the owners would finish their buildings to the sum provided or with the addition of a little more. For those who can provide 400,000 sesterces, and have to add 100,000 are content to be so bound, in the hope of completing the work while those who are burdened with the addition of a half, or more of the expense, lose hope, and declining further expenditure are forced to give up with broken fortune and spirit."—[From VITRUVIUS on ARCHITECTURE (Vol. II).—The Loeb Classical Library. Translator: Frank Granger.]

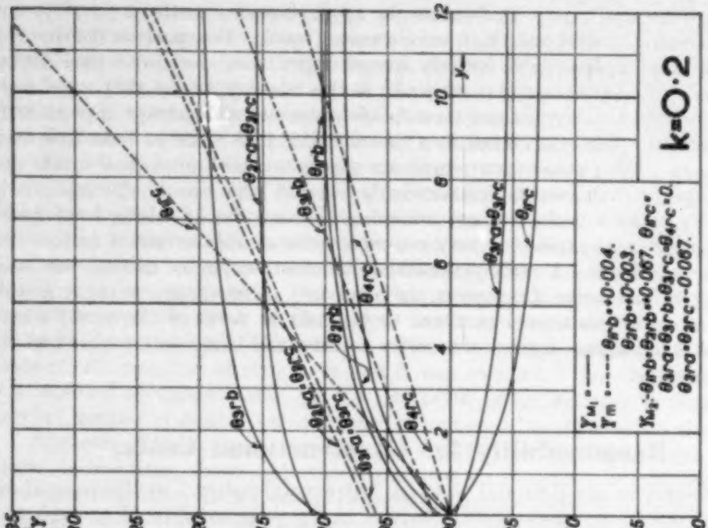
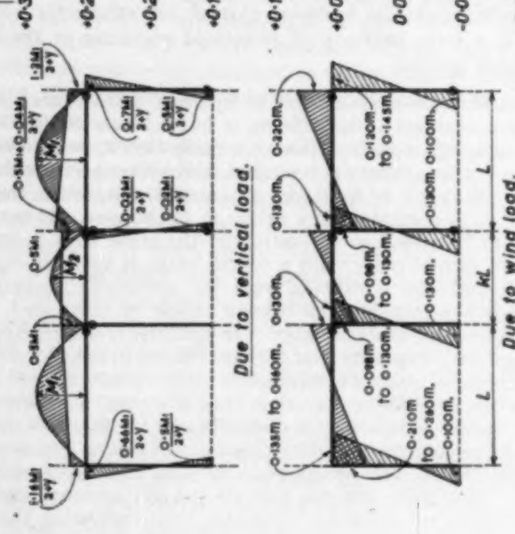


Chart No. 1.

See facing page.



RESULTANT MAXIMUM BENDING MOMENTS FOR WIND AND VERTICAL LOAD ON MEMBERS OF A TYPICAL STOREY.

Fig. 1.

The Analysis of Three-bay Frames by the Plastic-hinge Method.

By K. POOLOGASUNDRAM, Ph.D., A.M.I.C.E.

In an article in this journal in February 1959[1], expressions are developed for the application of the theory of plastic hinges[2] to a multiple-storey frame of three bays (*Fig.* 1) the ratio of the central span to the equal outer spans being variable. The object of the present article is to present suitable curves, based on these expressions, to enable the analysis of such frames to be speedily carried out. Expressions such as equations II in the previous article, suitable for graphical representation, can be derived by adopting the following assumptions and limiting values.

(1).—The length of each end span is L; the length of the shorter central span is kL. The moments of inertia I of all beams are uniform; the moments of inertia J of all columns in the same storey are also uniform. Therefore $y = \frac{Ih_r}{JL}$. Also $\frac{h_{r-1}}{J_{r-1}} = \frac{sh_r}{J_r}$; the limits of s are 0·66 and 1.

(2).—Vertical load on all beams may be either uniformly distributed or concentrated at the third-points. The maximum ordinate of the free bending-moment diagram in both of the end spans is M_1, and in the central span is M_2.

(3).—Values of m and $\bar{X}$ in the $(r + 1)^{th}$ storey with regard to the terms containing m are a times the values for the r^{th} storey; the limits of a are 0·75 and 1·0.

(4).—Values of m and $\bar{X}$ in the $(r - 1)^{th}$ storey with regard to the terms containing m are b times the values for the r^{th} storey; the limits of b are 1·25 and 1·0.

Values of Plastic-hinge Moments.

The limiting values for the plastic-hinge moments are given in formula (1), but differ slightly from those given in the previous article and are based upon research carried out by the author at Imperial College, London[3].

$$\left.\begin{array}{ll} \bar{X}_{1rb} = \bar{X}_{2rb} = 0{\cdot}13m + 0{\cdot}5M_1, & \bar{X}_{3rb} = 0{\cdot}22m + \dfrac{1{\cdot}2M_1}{2 + y} \\ \bar{X}_{2rc} = \bar{X}_{2ra} = 0{\cdot}13m, & \bar{X}_{3ra} = \bar{X}_{3rc} = 0{\cdot}13m - \dfrac{0{\cdot}02M_1}{2 + y} \\ \bar{X}_{1rc} = 0{\cdot}10m - \dfrac{0{\cdot}5M_1}{2 + y}, & \bar{X}_{4rc} = 0{\cdot}10m + \dfrac{0{\cdot}5M_1}{2 + y} \end{array}\right\} \quad . \quad (1)$$

---

1. E. W. Tokarski.—" Application of the Plastic-hinge Method to a Multiple-story Frame." " Concrete and Constructional Engineering ", February 1959.
   Errata.—In *Table* I in this article the value of the influence of $\bar{X}_{3rc}$ on θ_{3ra} should be $-\frac{1}{8}(2 + 2a + y)$ and the value of the influence of $\bar{X}_{4rc}$ on θ_{4rc} should be $-\frac{1}{8}\{2(a + b - 2) + y[sb + a - 4(1 + s)]\}$.—Ed.
2. A. L. L. Baker.—" The Ultimate Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames." 1956.
3. K. Poologasundram.—" An Analytical and Experimental Investigation of the Formation and Behaviour of Plastic Hinges in Prestressed and Reinforced Concrete Frames." University of London, Ph.D. Thesis, 1959.

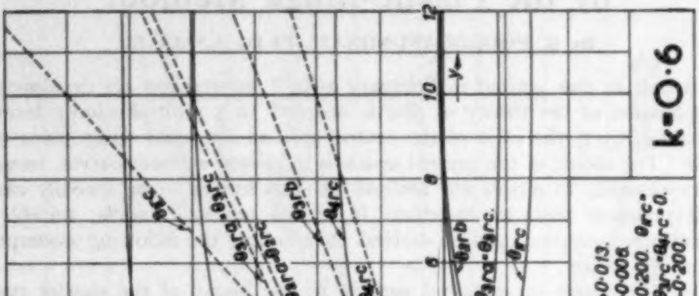


Chart No. 3.

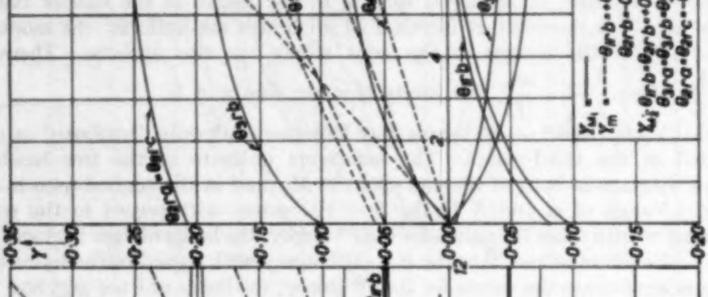


Chart No. 2.

Substituting these plastic-hinge moments in equations II and expressing the ratio $\frac{M_2}{M_1}$ as r_M and the ratio $\frac{m}{M_1}$ as r_m, formulæ (2) are obtained.

$$\left.\begin{aligned}
\theta_{1rb} &= \frac{M_1L}{EI}\left[\frac{0{\cdot}167y - 0{\cdot}25ky - 0{\cdot}5k + 0{\cdot}140}{2+y} + 0{\cdot}333kr_M + 0{\cdot}021kr_m\right] \\
\theta_{2rb} &= \frac{M_1L}{EI}\left[\frac{0{\cdot}167y - 0{\cdot}25ky - 0{\cdot}5k + 0{\cdot}12}{2+y} + 0{\cdot}333kr_M + (0{\cdot}0067 - 0{\cdot}021k)r_m\right] \\
\theta_{3rb} &= \frac{M_1L}{EI}\left[\frac{0{\cdot}213y + 0{\cdot}206}{2+y} + (0{\cdot}013y + 0{\cdot}013)r_m\right] \\
\theta_{2ra} &= \frac{M_1L}{EI}\left[\frac{0{\cdot}113y + 0{\cdot}25ky + 0{\cdot}5k + 0{\cdot}113}{2+y} - 0{\cdot}333kr_M + (0{\cdot}015y - 0{\cdot}021k + 0{\cdot}065)r_m\right] \\
\theta_{3ra} &= \frac{M_1L}{EI}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}015y + 0{\cdot}058)r_m\right] \\
\theta_{1rc} &= \frac{M_1L}{EI}\left[\frac{-0{\cdot}08y}{2+y} + 0{\cdot}03yr_m\right] \\
\theta_{2rc} &= \frac{M_1L}{EI}\left[\frac{0{\cdot}113y + 0{\cdot}25ky + 0{\cdot}5k + 0{\cdot}113}{2+y} - 0{\cdot}333kr_M + (0{\cdot}015y - 0{\cdot}022k + 0{\cdot}065)r_m\right] \\
\theta_{3rc} &= \frac{M_1L}{EI}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}015y + 0{\cdot}059)r_m\right] \\
\theta_{4rc} &= \frac{M_1L}{EI}\left[\frac{0{\cdot}1y}{2+y} + 0{\cdot}01yr_m\right]
\end{aligned}\right\} \quad . \quad (2)$$

Design Charts.

The general form of all the foregoing expressions is

$$\theta = \frac{M_1L}{EI}[Y_{M_1} + Y_{M_2}r_M + Y_m r_m],$$

where Y_{M_1}, Y_{M_2} and Y_m are functions of y for a given value of k. The charts accompanying this article give the values of these functions plotted against values of y from 0 to 12. Separate charts are given for values of k of 0·2, 0·4, 0·6, 0·8 and 1·0. Using these charts it is possible to calculate the rotations of the hinges easily and quickly as r_M and r_m are known quantities.

The object of the calculations is to make the rotations positive and to keep them within the permissible values. If the rotations obtained by using the curves are negative or are greater than the permissible values, the bending moment diagrams can be adjusted at certain sections, still keeping the adjusted moments in equilibrium. The additional rotations can be determined by applying the influence coefficients given in *Table* I in the previous article[1].

Example.—To determine the rotations of the hinges of an intermediate storey of a three-bay frame having the stated dimensions and subjected to the bending moments specified.

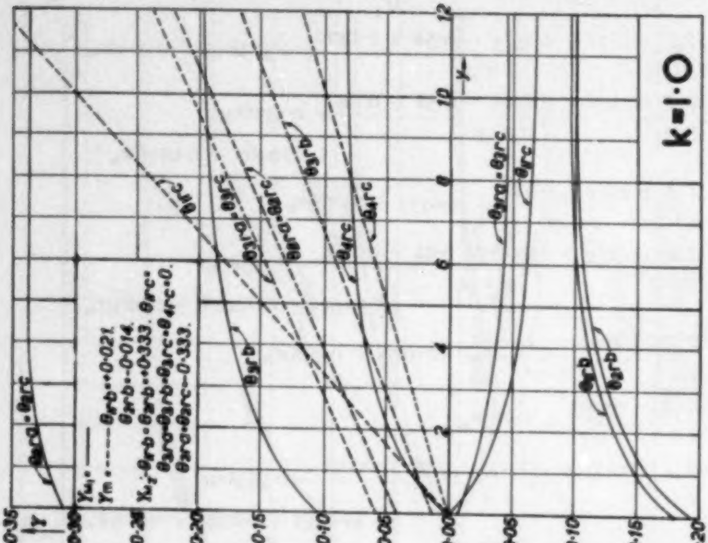


Chart No. 5.

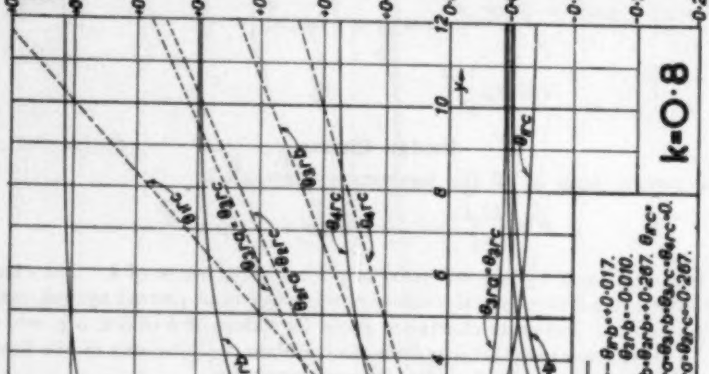


Chart No. 4.

End spans $L = 20$ ft.; central span $kL = 10$ ft.; hence $k = 0{\cdot}5$; storey-height $h = 12$ ft.

Free bending moments (at ultimate load):

$$\text{On outer beams } M_1 = 6{,}000{,}000 \text{ in.-lb.}$$
$$\text{On central beam } M_2 = 3{,}000{,}000 \text{ in.-lb.}$$

Free sway moment m due to wind $= 2{,}000{,}000$ in.-lb.

It is assumed that the values of a, b and s for the storeys above and below the storey considered are within the limits assumed in the charts. The crushing strength of works-cubes u_w at twenty-eight days is 4000 lb. per sq. in.; the equivalent elastic compressive stress in bending $c' = 0{\cdot}5u_w = 2000$ lb. per sq. in.

Assume the maximum bending moment M' on an end span to be approximately $0{\cdot}6 \times 6{,}000{,}000 = 3{,}600{,}000$ in.-lb. Since the section at which this bending moment occurs is within the elastic range, $I = \dfrac{M'n_1d}{c'}$ where n_1d is the depth to the neutral axis. Assume the columns to be 18 in. by 12 in., $d = 18$ in., and $n_1 = 0{\cdot}33$.

Then $I = \dfrac{3{,}600{,}000 \times 0{\cdot}33 \times 18}{2000} = 10{,}700 \text{ in.}^4$ and $J = \dfrac{12 \times 18^3}{12} = 5840 \text{ in.}^4$

Thus $y = \dfrac{Ih}{JL} = \dfrac{10{,}700 \times 12}{5840 \times 20} = 1{\cdot}1$. If E_c is approximately $500u_w$, $E_c = 500 \times 4000 = 2 \times 10^6$ lb. per sq. in.

Then $\dfrac{M_1L}{EI} = \dfrac{6{,}000{,}000 \times 20 \times 12}{2 \times 10^6 \times 10{,}700} = 0{\cdot}067$.

$$r_M = \frac{3{,}000{,}000}{6{,}000{,}000} = 0{\cdot}5, \qquad r_m = \frac{2{,}000{,}000}{6{,}000{,}000} = 0{\cdot}33.$$

Interpolating between the charts for $k = 0{\cdot}4$ and $0{\cdot}6$, the values of Y_{M_1}, Y_{M_2} and Y_m are obtained for each hinge, for a value of y of $1{\cdot}1$. The rotations of the hinges are computed in *Table* A.

TABLE A.

Hinge	Y_{M_1}	Y_{M_2}	Y_m	$\theta = \dfrac{M_1L}{EI}(Y_{M_1} + r_MY_{M_2} + r_mY_m)$	θ
1-*r*-*b*	−0·021	0·167	0·011	0·067(− 0·021 + 0·084 + 0·004)	0·0045
2-*r*-*b*	−0·028	0·167	−0·004	0·067(− 0·028 + 0·084 − 0·001)	0·0037
3-*r*-*b*	0·142	0	0·027	0·067(0·142 + 0 + 0·009)	0·0100
2-*r*-*a*	0·201	−0·167	0·071	0·067(0·201 − 0·084 + 0·024)	0·0094
3-*r*-*a*	−0·020	0	0·076	0·067(−0·020 + 0 + 0·025)	0·0003
1-*r*-*c*	−0·028	0	0·033	0·067(−0·028 + 0 + 0·011)	−0·0011
2-*r*-*c*	0·201	−0·167	0·071	0·067(0·201 − 0·084 + 0·024)	0·0094
3-*r*-*c*	−0·020	0	0·076	0·067(−0·020 + 0 + 0·025)	0·0003
4-*r*-*c*	0·035	0	0·011	0·067(0·035 + 0 + 0·004)	0·0027

It will be seen that the rotations of all the hinges are positive with the exception of 1-*r*-*c*. This rotation can be made positive by reducing the plastic moment of this section, but while so doing care must be taken to make any adjustments to the plastic moments at other sections which may be necessary in order to keep the distribution of bending moments in equilibrium with the applied loads.

Reduce $\bar{X}_{1rc}$ by $0{\cdot}1 \dfrac{M_1}{2 + y} = 0{\cdot}032\, M_1$ and, to maintain equilibrium, increase $\bar{X}_{4rc}$ by $0{\cdot}032\, M_1$ correspondingly. The resulting changes in the rotations may be determined by use of the influence coefficients in *Table* I(1). The additional rotations are

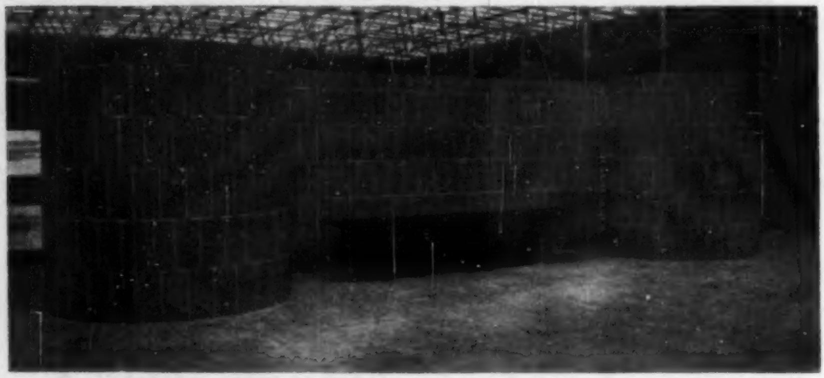
$\Delta\theta_{1rb} = \Delta\theta_{2rb} = 0$, $\Delta\theta_{3rb} = \Delta\theta_{1rc} = 0{\cdot}0024$, $\Delta\theta_{2ra} = \Delta\theta_{3ra} = \Delta\theta_{2rc} = \Delta\theta_{3rc} = 0{\cdot}0012$ and $\Delta\theta_{4rc} = -0{\cdot}0024$.

Adding these hinge-rotations to those obtained in *Table* A, the final values of rotation at the plastic hinges are

$$\begin{aligned} \theta_{1rb} &= 0{\cdot}0045, & \theta_{2rb} &= 0{\cdot}0037, & \theta_{3rb} &= 0{\cdot}0124, \\ \theta_{2ra} &= 0{\cdot}0106, & \theta_{3ra} &= 0{\cdot}0015, & \theta_{1rc} &= 0{\cdot}0013, \\ \theta_{2rc} &= 0{\cdot}0106, & \theta_{3rc} &= 0{\cdot}0015, & \theta_{4rc} &= 0{\cdot}0003. \end{aligned}$$

All the rotations are now positive and are reasonably small. Expressions for the maximum permissible rotations have been developed by the writer after detailed research and additional information on the formation and behaviour of plastic hinges with different types of reinforcement, etc., is also available[3].

An Architectural Congress.



The accompanying illustration shows an ornamental wall at the enquiry desk at the Sixth Congress of the International Union of Architects, which was held in London in July last. The wall was 24 ft. long with an enclosure 4 ft. in diameter at each end, and 8 ft. 3 in. high. It was built of "Spectra-glaze" blocks of a standard colour called "decorative blue". The blocks are of concrete with the surface impregnated with thermo-setting resin.

As announced in the number of this journal for April 1961, the theme of the Congress was "New Techniques and Materials—Their Impact on Architecture". Despite the subject, the wall illustrated, a similar structure at the book exhibition held in connection with the Congress, and some paving were the sole material representatives of the concrete industry. In the accompanying exhibition of photographs, however, concrete structures of many types predominated.

Included in the papers presented at the Congress was one by Pier Luigi Nervi entitled "Reinforced Concrete and Technical and Scientific Progress of the Architecture of Today and Tomorrow".

Concrete Construction at Trawsfynydd Nuclear Power Station.

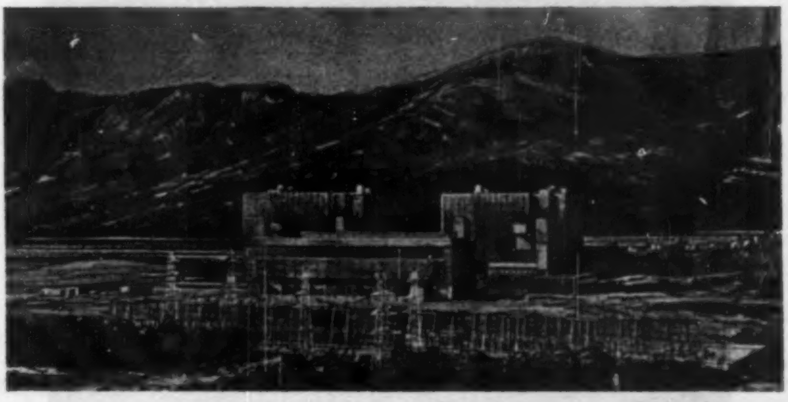


FIG. 1.

THE civil engineering work at the 500-megawatt nuclear power station (*Fig.* 1) in the course of construction for the Central Electricity Generating Board at Trawsfynydd, North Wales, is now well advanced. The station is on a hilly site about 600 ft. above sea level and adjacent to Trawsfynydd Lake. The principal structures are the two reactor buildings and the turbine house. The cooling-water pump house will contain six pumps which will be capable of circulating 500,000 gallons of water per minute, the water being drawn from the lake and returned thereto for cooling by natural means. A labour camp near the site has accommodation for about a thousand men.

The arrangement of the structures is

Fig. 2.

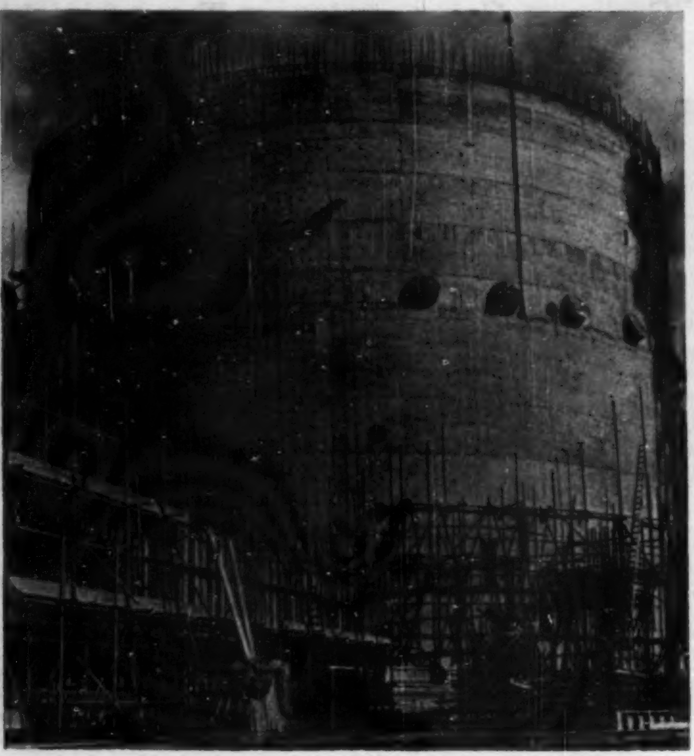


Fig. 3.

reasonably compact. There are three principal areas, namely the reactor area, fuel-disposal area, and turbine house area. Around the site there is a main access road, and therefore buildings requiring access from the public highway are located near this road so that there is no need for vehicles to enter the reactor or fuel-disposal areas.

Foundations.

The original site undulated by about 50 ft. and the quantity of excavation required to level the site and for the foundations was about 450,000 cu. yd. The ground is mainly rock, which is a silicious medium-grained greywacke in the Rhinog grit series of the Cambrian system. The rock was overlain in parts by a glacial moraine and much peat, both of which materials contained boulders. The rock, however, is at convenient levels to enable most of the main foundations to bear on it; as a result there is no likelihood of differential settlement occurring between associated parts of the plant.

The foundations of the reactors comprise a raft and extend 25 ft. below the finished ground level of the site. The raft was cast in sections (*Fig.* 2). Thermocouples embedded in the base of the reactors were installed on precast pillars (*Fig.* 5) which were retained in place by a steel-angle frame as shown in the illustration.

The foundations for the rail-track for a 400-ton Goliath crane also bear on the rock which is in places 20 ft. below rail level. The concrete in this foundation was placed by pump.

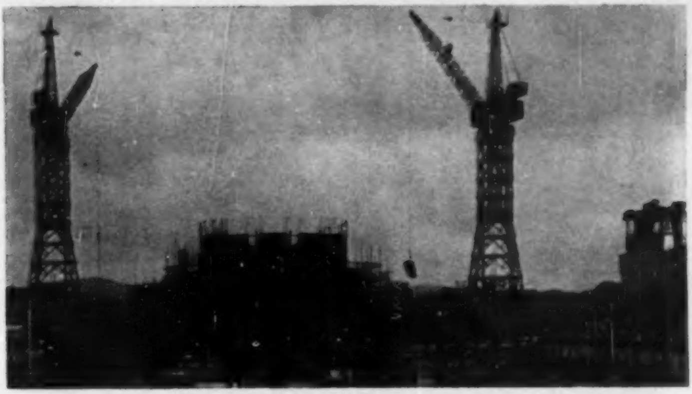


Fig. 4.

Foundations for some of the temporary buildings, such as workshops, are on consolidated filling, but special foundations, such as those for test beds and furnaces, are taken down to undisturbed moraine.

Reactor Buildings.

Each reactor building, of which there are two, contains a reactor and six boilers (or heat-exchangers), and are each 298 ft. long, 180 ft. wide, and 179 ft. high, with a substructure extending 17 ft. 9 in. below the finished ground level. The buildings and shield-walls are of reinforced concrete, except the roof frames, crane beams and parts of the boiler houses which are of steel. The cladding is mainly of precast concrete panels, but parts of the boiler houses are of cast-insitu concrete.

The biological shield-walls are 10 ft. to 10 ft. 6 in. thick and form a cylinder (*Fig.* 3) of 70 ft. diameter. Their height

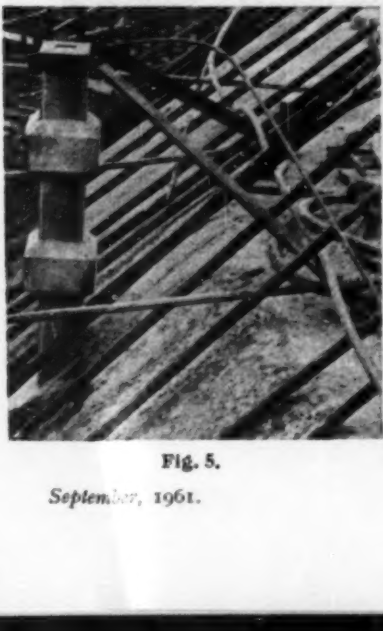


Fig. 5.

Fig. 6.

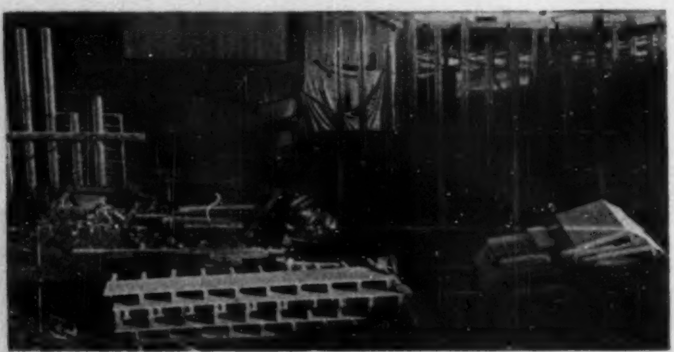


Fig. 7.

is 93 ft. above the foundations. The top slab over the reactors will be 11 ft. 6 in. thick, and will comprise precast reinforced concrete beams combined with cast-insitu concrete. The inner face of the shield-wall will be cooled by air flowing in a cavity between twin steel plates, one of which forms permanent shuttering for the wall.

The boilers, which are 18 ft. in diameter and 108 ft. high, are supported on reinforced concrete frames carried on plinths 31 ft. above the ground. The reinforced concrete walls around the boilers are 100 ft. high and, for shielding purposes, are from 12 in. to 18 in. thick. The walls are being built after the pipes connected to the boilers are installed.

The site of each reactor building is commanded by two mono-tower cranes (*Fig.* 4) which can travel on short lengths of parallel tracks, and can lift 5 tons at a radius of 100 ft.

Cooling Pond.

The pond for cooling irradiated fuel elements is parallel to the reactor buildings and is 335 ft. long, 35 ft. wide and 20 ft. deep. The fuel elements are passed to the pond through massive reinforced concrete chutes. The thickness of the bottom and walls of the pond is generally

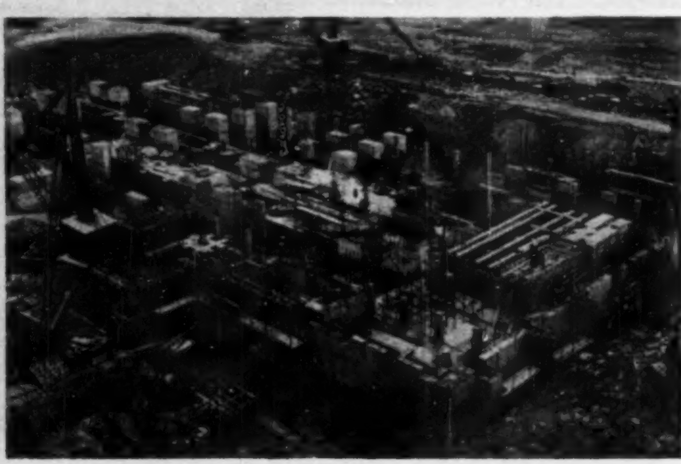


Fig. 8.

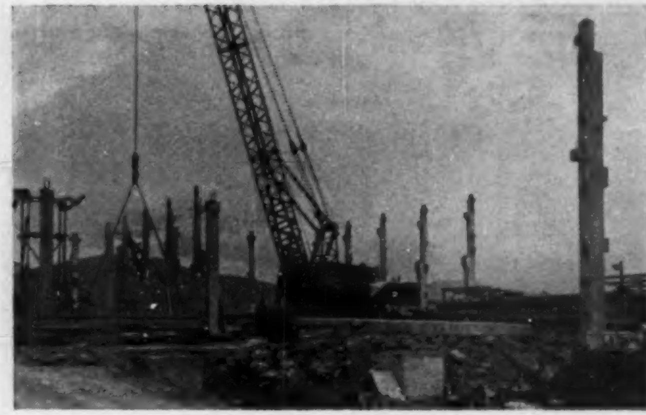


Fig. 9.

about 3 ft. The shuttering for the walls (*Figs.* 6 and 7) comprises large timber panels, with plywood lining, some of which are 18 ft. by 7 ft. After each use the panels are repaired, if necessary, at the site of the pond. They are lifted by means of a mobile crane on endless tracks and, if required at the other end of the pond, are transferred thereto on a low-loader. The walls are constructed in lengths equal to that of the panels of shuttering and P.V.C. water-bars are inserted across the horizontal and vertical joints. The vertical joints ensure that shrinkage is taken up at predetermined positions along straight lines with the edges of the joint suitably treated.

Turbine Hall.

The turbine hall will contain four turbo-generators arranged transversely. The main building, which is 370 ft. long, 140 ft. wide and 96 ft. high, is of steelwork, but the foundations, the plinths supporting the turbo-generators, the basement walls, circulating ducts, and some floors and roofs in the mechanical and electrical annex are of reinforced concrete. *Fig.* 8 shows the cast-insitu lower part of the walls in the course of construction. The

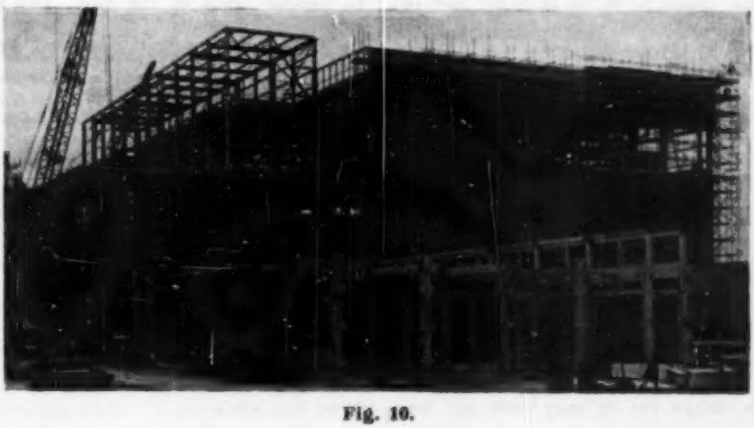


Fig. 10.

Fig. 11.

columns of the workshop attached to the turbine hall were precast on the site and were erected by excavators used as cranes (*Fig.* 9).

The cladding of the walls of the turbine hall and workshop are of precast exposed-aggregate slabs (*Fig.* 10), which are described in a subsequent paragraph.

Pump House.

The cooling-water pump house is a reinforced concrete structure extending 30 ft. below the normal level of Trawsfynydd Lake, on the shore of which it is situated. In plan, the structure is 250 ft. long and 110 ft. wide. Two mild steel pipes, about 10 ft. in diameter and encased in concrete, extend from the pump house to the turbine hall.

The outlet works are of reinforced concrete and are 80 ft. long; the width at the narrowest part is 42 ft., fanning out to 90 ft. Automatic safety gates, penstocks, and dam-boards are provided. The gates are intended to eliminate risk of flooding in the unlikely event of a break occurring in the discharge pipes.

Concrete.

The total amount of concrete in the works is about 200,000 cu. yd. Most of the concrete is made at a central batching and mixing plant (see *Fig.* 11 and on the right in *Fig.* 4) from which it is distributed in $1\frac{1}{4}$-cu. yd. tipping-hopper trucks to various parts of the site. For the turbine-hall basement, the pump house, the Goliath-track foundations, and some

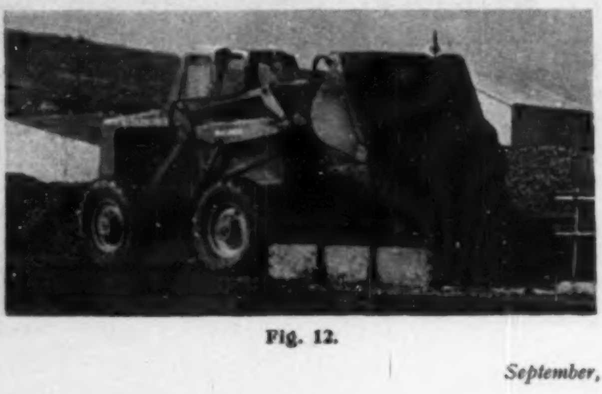


Fig. 12.

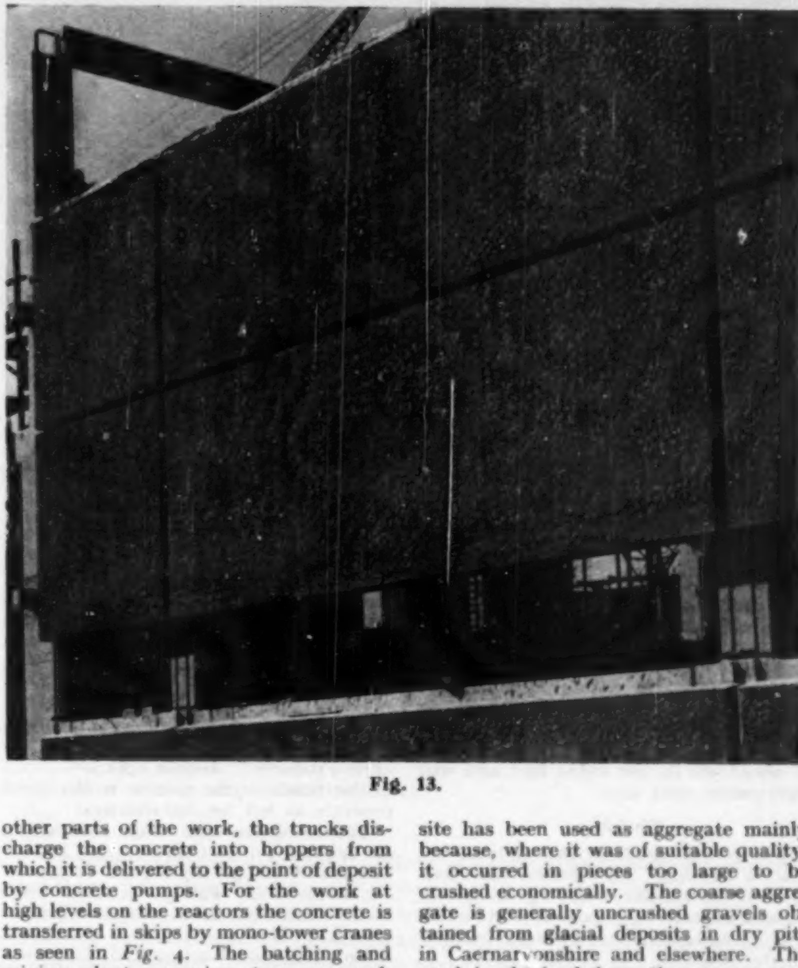


Fig. 13.

other parts of the work, the trucks discharge the concrete into hoppers from which it is delivered to the point of deposit by concrete pumps. For the work at high levels on the reactors the concrete is transferred in skips by mono-tower cranes as seen in *Fig.* 4. The batching and mixing plant comprises two 1-cu. yd. drum-type mixers and has produced up to 2800 cu. yd. of concrete in a week. Most concrete is compacted by internal vibrators which are generally 4-in. high-frequency machines. Curing in dry weather is by means of saturated hessian.

The aggregates are stored in the open in radial compartments and are transported by a mobile shovel to the boot of the belt-conveyor (*Fig.* 12), which conveys the material to the storage bins of the batching plant. None of the rock on the site has been used as aggregate mainly because, where it was of suitable quality, it occurred in pieces too large to be crushed economically. The coarse aggregate is generally uncrushed gravels obtained from glacial deposits in dry pits in Caernarvonshire and elsewhere. The sand is obtained from the same source by screening the detritus. Some oversize material is crushed.

Several grades of concrete are produced for various purposes. Ordinary structural concrete is specified to have a crushing strength of 5000 lb. per square inch at twenty-eight days. For the shield-walls and similar work, concrete having a guaranteed dry-density is required, the specified strength required being an average of 4250 lb. per square inch and not less than 3000 lb. per square inch;

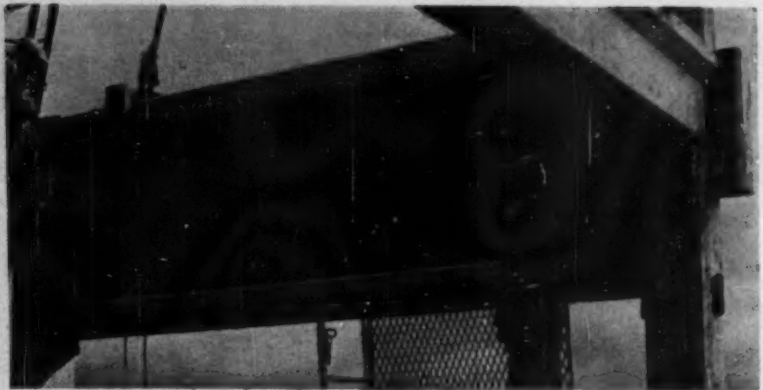


Fig. 14.

the actual average strength of the concrete supplied is about 5000 lb. per square inch. With gap-graded rounded gravel aggregate, a dry density of 144 lb. per cubic foot is obtained with a water-cement ratio of 0·5. Stones of ¾-in. size are omitted from the aggregate. In the concrete of guaranteed density, as also in mass concrete in the pump house which has to be watertight, pulverised-fuel ash obtained from the generating station at Bold, Lancs., is incorporated. About 800 tons of heavy concrete is to be made; this concrete is expected to have a density of about 360 lb. per cubic foot and will incorporate steel shot.

Finishes.

The general finish of the walls of the reactor buildings, the turbine hall, and some of the ancillary buildings comprises precast exposed-aggregate concrete slabs (*Fig.* 13) and are made in a factory installed on the site.

Some of the slabs are 25 ft. long and up to 8 ft. wide. The slabs are supported on corbels on the columns of the buildings, the method of attachment depending on whether the columns are of steel or of precast concrete as in *Fig.* 14. During erection, the slabs are drawn tightly against the supports by jigs attached to sockets embedded in the concrete. The face of the slabs is a crushed local grey granite, which is exposed by spraying with water and brushing with a soft broom while the concrete is still green. The crushed stone is a mixture of Arenig granite and Pengwern and Gwydyr granites. The backing concrete is ordinary concrete of structural grade with ¾-in. aggregate. In some of the buildings, the inner faces of the slabs are lined with wood-wool for thermal insulation purposes; the lining is embedded in the slabs at the time of manufacture. The wood-wool lining is seen in *Fig.* 14, which shows a view from the inside of a reactor building. [A description of the manufacture of the exposed-aggregate slabs is given in "Concrete Building and Concrete Products", August 1961.]

The finish to the reactor is fair-faced concrete as left by the shutters.

The designers and main contractors are Atomic Power Constructions Ltd., a consortium comprising Messrs. Crompton Parkinson Ltd., Fairey Aviation Ltd., Messrs. International Combustion (Holdings) Ltd., Messrs. Richardson Westgarth & Co. Ltd., and Nuclear Civil Constructors Ltd. The firm last named are the designers and contractors for the civil engineering work and comprise Messrs. Trollope and Colls Ltd., and Messrs. Holland & Hannen and Cubitts Ltd. The consulting architect to Nuclear Civil Constructors Ltd., is Sir Basil Spence and the consulting engineers are Messrs. Mott, Hay & Anderson. The sub-contractors include Pierhead Ltd., which firm supplied some prestressed precast slabs for floors in the turbine hall and prestressed beams for other buildings.

The Shearing Resistance of Beams by the Load-factor Method.

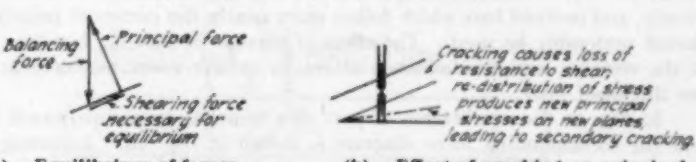
By C. ERDEI, B.Sc.Eng. (Hungary).

THE load-factor method is based on the principle that every part of a structure carrying a particular load should have the same factor of safety. The required factor of safety against shearing is not always achieved when the current methods of calculating shearing resistance are applied. Failure may occur before the calculated ultimate load is reached if the distribution of shearing reinforcement is unsatisfactory although the amount may be sufficient, or if the amount of main tension reinforcement is sufficient but if the bars do not extend beyond the position at which an inclined crack can start.

Notation and Definitions.

b: Width of beam. d_1: Effective depth of beam.
δ: Original inclination of reinforcement to the axis of the beam.
ω: Angle between the normal to a principal crack and the axis of the beam.
α: Inclination of reinforcement after opening of crack.
h: Horizontal projection of inclined crack.
d_n: Depth of compression zone.
Q, Q_c: Total shearing resistance, and shearing resistance of compressive zone respectively.
M: Total bending resistance.
Q_s: Force at failure in reinforcement provided to resist shearing.
T: Force at failure in main tension reinforcement.
f_u: Tensile strength of steel.
p_{cb}: Safe compressive stress in concrete due to bending.
p_{st}: Safe tensile stress in shear reinforcement.
u_{ce}: Mean compressive bending stress in concrete at failure.
u_{cb}, u_{cs}: Bearing and shearing strengths of concrete.
β: $\dfrac{u_{ce}}{u_{cs}}$. γ: $\dfrac{\beta}{2b \, . \, u_{cs}}$.
A_{st}: Area of main tension reinforcement.
A_{sv}: Area of stirrups.
A_{si}: Area of inclined reinforcement.

The common assumption that the effects of shearing can be treated separately from those of bending has been shown to be incorrect. In reinforced concrete, as in homogeneous materials, the existence of stresses on two mutually perpendicular planes, caused by external loads, gives rise to principal stresses acting on principal planes. It has been shown that cracks due to shearing follow the directions of the principal stresses which would exist in a homogeneous material. These are here termed "principal cracks" (see *Fig.* 2a).



(a).—Equilibrium of forces. **(b).—Effect of cracking on principal stresses.**

Fig. 1.

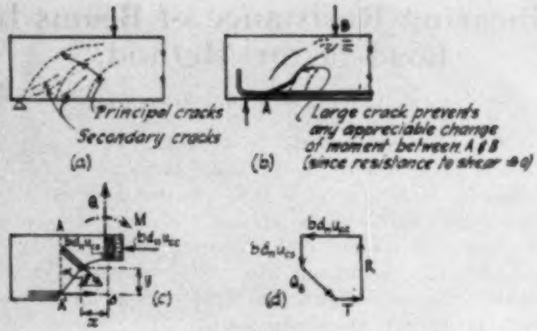


Fig. 2.—Cracking and Stresses at Failure.

After the occurrence of the principal cracks, a second system of cracks (here termed " secondary cracks ") is formed (*Fig.* 2a). The extent of the secondary cracks depends on the additional stresses induced by the reinforcement, which at this stage has to hold together the concrete previously split by the principal cracks. The more closely the reinforcement follows the direction of the principal stress, the smaller is the importance and the extent of the secondary cracks; in theory, no secondary cracks occur if the reinforcement conforms exactly to the direction of the principal tensile stress.

Stirrups and Inclined Bars.

Fig. 1a shows that if a principal force is balanced by a force in an arbitrary direction at an angle smaller than 90 deg., measured from the normal to the plane on which the force is acting, an additional shearing force must be present if the system is to be in equilibrium. This well-known result is based on the assumption that the strains in the concrete and the steel are equal. After the concrete has cracked along the principal planes no shearing force can operate along the crack. An immediate redistribution of internal forces and stresses occurs, which causes an angular displacement of the principal planes and the formation of secondary cracks along the new lines of principal stress (*Fig.* 1b).

The extent of these secondary cracks increases as the load increases; it is possible for one or more to unite with a major primary crack, the resulting " rupture-line " leading possibly to a premature failure in bending. (From *Fig.* 2b it is obvious that if the resistance of stirrups to bending is neglected the longitudinal reinforcement at A must resist the bending moment which occurs at B.) Vertical stirrups cause the inclined cracks to extend longitudinally, and inclined bars which follow more nearly the curves of principal stress, should preferably be used. The effect of stirrups in resisting bending, the pitch of the stirrups, and the influence of cracks on the resistance-moment diagram are discussed later.

In *Fig.* 2c the equilibrium of part of a beam at a principal crack is shown, and the corresponding force diagram is shown in *Fig.* 2d. Assuming that the shearing force exceeds the shearing strength of the concrete, the basic equations of equilibrium lead to formulæ (1) to (3).

$$M = bd_n u_{cc}(d_1 - \tfrac{1}{2}d_n) + Q_s(x.\sin\alpha - y.\cos\alpha) \quad . \quad . \quad (1)$$

$$Q = bd_n u_{cs} + Q_s \sin\alpha \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$T = bd_n u_{cc} - Q_s \cos\alpha \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

It follows from *Fig.* 2c that the distance between the inclined bars is as important as the area of the bars. At least one bar (or in the case of stirrups, a pair of bars) should pass through each inclined crack, and this means that the spacing of the stirrups or inclined bars is limited by the horizontal projection h of the inclined cracks. From this requirement the intensity of the force Q_s induced in the shearing reinforcement can be computed. The inclinations and lengths of the inclined cracks can be obtained either accurately by computing principal stresses and planes, or approximately by comparison with the known principal stresses and planes of similar test beams with similar loading; the latter method is usually more convenient and sufficiently accurate. Since the positions of the cracks cannot be predicted accurately, limiting values can be obtained for Q_s and T, depending on the relative positions of the cracks and the reinforcement. The greatest value of Q_s, and the least value of T, are obtained if the shearing reinforcement is located at the outer end of the crack (adjacent to the main reinforcement), since it must then resist both that part of the shearing force Q which cannot be resisted by the compressive zone, and the greatest proportion of the bending moment. Conversely, the least value of Q_s and the greatest value of T occur if the shearing reinforcement is located at the inner end of the crack. In the latter case, if stirrups are provided, no bending moment is resisted by the stirrups and the main tension reinforcement at A′ must resist the bending moment at section BB′ which is obviously greater than that at section AA′ (*Figs.* 2b and 3). For these two limiting cases the following analysis applies.

Greatest Value of Q_s (Least Value of T).—Formula (1) becomes

$$M = bd_n u_{cc}(d_1 - \tfrac{1}{2}d_n) + Q_s h \sin\alpha \quad . \quad . \quad . \quad (1a)$$

Q is as given by formula (2), from which $d_n = \dfrac{Q - Q_s \sin\alpha}{bu_{cs}}$; substituting in (1*a*) and simplifying,

$$Q_{s(max.)} = \frac{2\gamma Q - \beta d_1 + h + \sqrt{(\beta d_1)^2 - 4\gamma(M - hQ) - 2\beta d_1 h + h^2}}{2\gamma \sin\alpha} \quad . \quad (4)$$

Greatest Value of T (Least Value of Q_s).—From formula (1),

$$M = bd_n u_{cc}(d_1 - \tfrac{1}{2}d_n) - Q_s(d_1 - d_n)\cos\alpha \quad . \quad . \quad . \quad (1b)$$

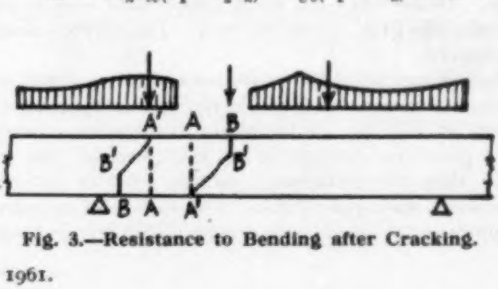


Fig. 3.—Resistance to Bending after Cracking.

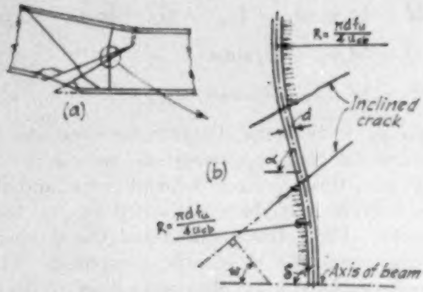


Fig. 4.—Distortion of Bar across a Crack.

Since Q is given by formula (2), d_n is as in the foregoing; hence from (1*b*)

$$M = \beta(Q - Q_s \sin\alpha)\left(d_1 - \left[\frac{Q - Q_s \sin\alpha}{2bu_{cs}}\right]\right) - Q_s \cos\alpha\left(d_1 - \left[\frac{Q - Q_s \sin\alpha}{bu_{cs}}\right]\right),$$

from which $Q_{s(min.)}$

$$= \frac{K(2\gamma Q - \beta d_1) + \sqrt{\left(\frac{4\gamma Q}{\beta}\left[Q\frac{\gamma}{\beta} - d_1\right] + d_1^2\right)\cos^2\alpha - (4\gamma M - [\beta d_1]^2)L\sin\alpha}}{2\gamma L \sin\alpha} \qquad (4a)$$

where $$K = \sin\alpha + \frac{\cos\alpha}{\beta} \text{ and } L = \sin\alpha + \frac{2\cos\alpha}{\beta}.$$

Also $$T_{max.} = \beta(Q - Q_s \sin\alpha) - Q_s \cos\alpha \qquad . \quad . \quad . \quad (3a)$$

which is $\beta(Q - Q_s)$ for vertical stirrups.

Theoretical considerations show that the final slope of an inclined bar, after its yield point is reached in a widening crack, is between δ and ω. In *Fig.* 4a is shown the calculated shape of the centre-line of the bar after yield, disregarding the high bearing stress induced in the concrete by the acute change of direction of the bar. In practice the deformation of the bar must approximate to that shown in *Fig.* 4b; thus the greater of the two angles δ and ω should be allowed for in computing $T_{max.}$, and a value intermediate between δ and ω would be reasonable for computing $Q_{s(max.)}$. Most investigators agree that a suitable value for β is between 5 and 6.

Since the compressive zone is always subjected to combined shearing and compression, the greatest compressive stress occurs on a plane inclined to the axis of the beam. Hence when the whole compressive zone is required to resist bending, a reduced value of u_{cc} should be used. Theoretical calculations indicate that $0{\cdot}8u_{cc}$ is adequate.

Only the limiting spacing of the reinforcement is considered in the foregoing. For practical reasons, such as ease of placing and compacting the concrete, it is usually desirable to place the reinforcement at the greatest permissible spacing and the formulæ given are convenient for this purpose. On the other hand, a spacing smaller than the maximum produces a better distribution of stress and should be adopted when practicable. This case is more indeterminate than the limiting conditions considered in the foregoing, but the problem can be solved

Fig. 5.—Equivalent Resistance of Reinforcement.

in the same way by considering the plastic properties of the reinforcement, using the resultant of the oblique forces produced by each separate bar. When the value of δ is obtained in this way Q_s can be calculated by considering the forces $Q_1, Q_2, Q_3 \ldots Q_n$ to be proportional to the areas $A_1, A_2, A_3 \ldots A_n$ (*Fig.* 5). This assumption can be made only if ductile steel is used, and even so restricted strains should be allowed.

Application to Design.

Contrary to the assumption in the elastic theory, a plane section does not remain plane after bending, but comprises a curve A′B′ and a plane B′B joining at B′ (*Fig.* 3). The amount of main tension reinforcement calculated for the relevant (and fictitious) elastic cross-sections should extend beyond the non-planar sections, as shown in the following.

Since $Q < b.d_n u_{cs}$ and $M = bd_n u_{cc}(d_1 - \tfrac{1}{2}d_n)$, $\dfrac{M}{Q} > \beta d_1 - \dfrac{\beta^2 Q}{2bu_{cc}}$ (Case *A*).

The resistance-moment diagram is plotted in the usual manner, and is then extended by distances h equal to the horizontal projections of the lengths of the cracks (*Fig.* 6).

If $\dfrac{M}{Q} < \beta d_1 - \dfrac{\beta^2 Q}{2bu_{cc}}$ (Case *B*), with stirrups, formulæ (4*a*) and (3*a*) could be used to compute the value of $T_{max.}$, but these give the same result as the simple formulæ derived for the case of pure bending.

Consider the case of simple bending in which the resistance of the compressive zone to shearing equals the shearing force Q. Then $\dfrac{bd_n u_{cc}}{\beta} = Q$; $d_n = \dfrac{\beta Q}{bu_{cc}}$.

In Case A,
$$M = \beta Q d_1 - \frac{\beta^2 Q^2}{2bu_{cc}} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Since $T = bd_n u_{cc}$, $T = \beta Q$.

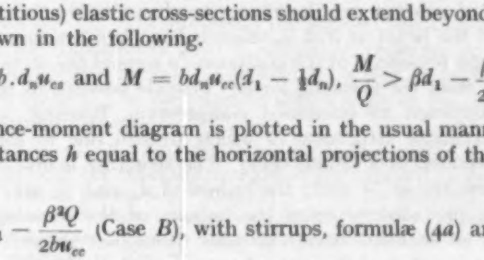


Fig. 6.—Adjustment of Bending Moment.

Now consider the case of combined bending and shearing. Substituting the bending moment given by (5) in (4a), and simplifying,

$$Q_{s(min)} = \frac{2\gamma Q - \beta d_1 + \sqrt{\beta^2 d_1^2 - 4\beta d_1 \gamma Q + 4\gamma^2 Q^2}}{2\gamma} = 0$$

(that is, no shearing reinforcement is required) and $T = \beta(Q - Q_s) = \beta Q$, as before.

If $M = 0$, then $Q_{s(min)} = \frac{2\gamma Q - \beta d_1 + \sqrt{\beta^2 d_1^2}}{2\gamma} = Q.$

Similarly in Case B, $T = bd_n u_{cc}$ which is equal to the value calculated by the formula for simple bending. Consequently the simple method of plotting the resistance-moment diagram shown previously can still be used and the shearing reinforcement can be obtained by deducting the shearing resistance Q_c of the compressive zone from the total shearing force and resisting the rest of the shearing force by stirrups. The value of Q_c is $\frac{T}{\beta}$ and the area of stirrups A_{sv} is given by

$$A_{sv} = \frac{Q - Q_c}{2f_u} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The design of a beam can therefore be reduced to the following simple procedure. Design the beam as if it is subjected to pure bending only. Then calculate the shearing resistance of the compressive zone of the section thus obtained, and deduct this from the shearing force. Provide stirrups to resist the balance.

Columns subjected to combined compression, bending, and shearing, or statically-indeterminate structures in which stresses due to shrinkage are considered, can be treated in a similar way. The structure is first designed to resist only the eccentric thrust or pull; the values of d_n and Q_c are then determined and stirrups are provided to resist the balance of the shearing force.

In the case of inclined bars, the same principles should be followed. It follows from *Fig.* 2c that if the line of action of Q_s is chosen so that it passes through the point O, this force has no influence on the position of the neutral axis [formula (1)]. Hence the cross-section can be designed for bending (or combined bending and compression or tension) and the effect of shearing can be considered separately by balancing $Q - Q_c$ with a force at an angle α to the axis of the beam. In algebraic terms, $Q_s = \frac{Q - Q_c}{\sin \alpha}$ and $A_{st} = \frac{Q - Q_c}{f_u . \sin \alpha}$. (7)

Thus Q_s becomes greater and T becomes smaller than in the equivalent case when stirrups are used. Since T can be reduced even to zero, this can be considered as a limiting case in which no cracks can open. It is generally accepted that the depth to the neutral axis does not exceed about $0{\cdot}5d_1$, that is

$$\frac{f_u}{bd_1 u_{cc}}[A_{st} + A_{st} \cos \alpha] < 0{\cdot}5.$$

Since ultimate stresses are not generally used directly in design, allowable working stresses and load-factors should be inserted in the appropriate formulæ to conform to relevant regulations. In particular, the simplifications made in deriving formulæ (6) and (7) are valid only if the stress f_u is reduced by 20 to 25 per cent. to allow for the effect of the bending moment on the web reinforcement.

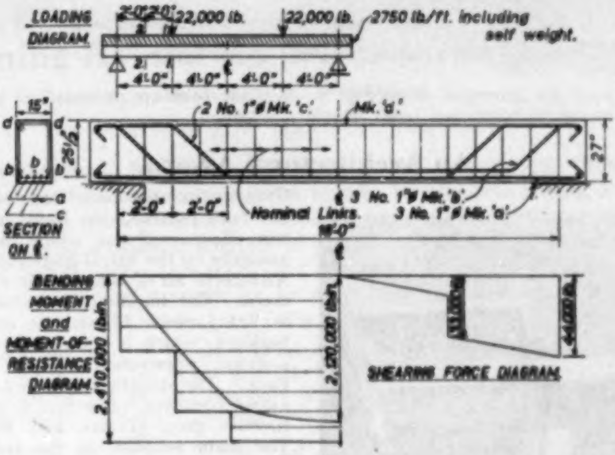


Fig. 7.

In formulæ (6) and (7), if Q represents the applied shearing force and Q_c the safe resistance of the compressive zone, f_u should be replaced by the safe tensile stress p_{st} in the shear reinforcement. Likewise, u_{cc} should be replaced by the safe mean compressive stress p_{cm} in the concrete which is about $\frac{1}{3}u_{cc}$ where u_{cc} is two-thirds of the crushing strength.

EXAMPLE.—Design the shear reinforcement for the beam shown in *Fig.* 7 if p_{st} is 20,000 lb. per sq. in. and the crushing strength of the concrete is 3000 lb. per sq. in. at 28 days (p_{cb} = 1000 lb. per sq. in.). Assume $\beta = 6$. To resist the positive bending moment the reinforcement required is seven 1-in. bars, but eight are provided since two of the bent-up bars are not fully effective in resisting bending moment (see the bending moment diagram in *Fig.* 7).

At centre of supports: Applied shearing force $Q = (2750 \times 8) + 22{,}000 = 44{,}000$ lb.

Section 1—1.—

Applied $M_1 = (44{,}000 \times 4 \times 12) - (2750 \times 4^2 \times 12 \times \frac{1}{2}) = 1{,}848{,}000$ lb.-in.

Applied $Q_1 = 44{,}000 - (2750 \times 4) = 33{,}000$ lb.

$\dfrac{M_1}{bd_1^2 p_{cb}} = \dfrac{1{,}848{,}000}{15 \times 25{\cdot}5^2 \times 1000} = 0{\cdot}190$; hence from formulæ* based on those in B.S. Code No. 114, $a_1 = 0{\cdot}848$ and $n_1 = 0{\cdot}344$. Since $p_{cm} = \frac{1}{3} \times \frac{2}{3} \times 3000 = 667$ lb. per sq. in.,

$$\text{Safe } Q_{c1} = \frac{T}{\beta} = \frac{bd_n p_{cm}}{\beta} = 15 \times 0{\cdot}344 \times 25{\cdot}5 \times 667 \times \tfrac{1}{6} = 14{,}600 \text{ lb.}$$

From formula (7) with bars bent up at 45 deg., and p_{st} substituted for f_u,

$$A_{st_1} = \frac{33{,}000 - 14{,}600}{20{,}000 \times 0{\cdot}707} = 1{\cdot}30 \text{ sq.-in., say two 1-in. bars.}$$

Similarly for Section 2—2.—

$M_2 = (44{,}000 \times 2 \times 12) - (2750 \times 2^2 \times 12 \times \frac{1}{2}) = 990{,}000$ lb.-in.

$Q_2 = 44{,}000 - (2750 \times 2) = 38{,}500$ lb.

$\dfrac{M_2}{bd_1^2 p_{cb}} = \dfrac{990{,}000}{15 \times 25{\cdot}5^2 \times 1000} = 0{\cdot}102$; hence $a_1 = 0{\cdot}917$ and $n_1 = 0{\cdot}167$. As at

$$^*a_1 = \frac{l_a}{d_1} = \tfrac{1}{2}\left(1 + \sqrt{1 - \frac{3M}{bd_1^2 p_{cb}}}\right) \quad \text{and} \quad n_1 = \frac{d_n}{d_1} = 1 - \sqrt{1 - \frac{3M}{bd_1^2 p_{cb}}}.$$

Section 1—1, $Q_{c2} = 15 \times 0{\cdot}167 \times 25{\cdot}5 \times 667 \times \frac{1}{6} = 7100$ lb. and

$$A_{st_2} = \frac{38{,}500 - 7100}{20{,}000 \times 0{\cdot}707} = 2{\cdot}22 \text{ sq. in., say three 1-in. bars.}$$

The bars are arranged as in *Fig.* 7. Anchor bars are provided at the ends of the bars owing to restricted bond length available.

An Architectural Award.

Once again a reinforced concrete building has been awarded the London Architecture Bronze Medal which is awarded annually by the Royal Institute of British Architects for a building of exceptional merit. The building is a block of flats in St. James's Place; the rear of the building, which is shown in the accompanying illustration, overlooks Green Park. The structure is basically a reinforced concrete frame but is faced with Baveno grey granite and blue brick. The main problem in the architectural design is harmonising a structure of modern appearance with adjacent eighteenth-century buildings.

The floors, which are of cast-insitu construction, are 7½ in. thick, thereby allowing the solid partitions to be re-positioned at any time. The columns are of H-section and provide ducts for services. The edge beams, which are wide and shallow, support the cantilevered balconies; the soffits of the beams are at ceiling level. Stability is provided mainly by the reinforced concrete lift-walls and stair-wells. The exposed concrete of the penthouse is white with a boarded finish, and the requirements of strength and colour are obtained by a 1 : 1½ : 3 mixture, the cement portion being a 1 : 3 mixture of white cement and ordinary Portland cement.

The architects are Messrs. Denys Lasdun & Partners and the consulting engineers are Messrs. Ove Arup and Partners. The contractors were Messrs. J. Jarvis & Sons Ltd.

Production of Aggregates.

The production of building and concreting sands and gravel aggregates in Great Britain in 1960 was, according to "Sand and Gravel Production 1959–1960" (Her Majesty's Stationery Office; price 1*s*. 3*d*.), 64,500,000 cu. yd., which is 11 per cent. higher than in 1959. In general, output has increased in all regions excepting Mid-Anglia where the output in 1959 was abnormal because of the demand for aggregates used in the construction of the motorway M1. In view of the large increase in the use of aggregates required for the increasing amount of construction work, it is estimated that the demand for sand and gravel will continue to increase, although not necessarily as rapidly as in the past two years, and that the total demand in England and Wales may reach 73,000,000 cu. yd. by 1970.

A Practical Comprehensive Method of Designing Reinforced Concrete Sections.—IV.*

By J. RYGOL, B.Sc.(Eng.).

LOAD-FACTOR METHOD (*continued*).

American Practice.—In the American report quoted on page 275 of this journal for August 1961, the ratio α/n is given as 0·85 for concrete having a cylinder strength of 5000 lb. per square inch. According to recent tests,(3) this ratio applies to concrete having a cylinder strength of 4000 lb. per square inch (that is a cube strength of 5333 lb. per square inch), and is reduced by 0·05 for each 1000 lb. per square inch of cylinder strength (or for each 1333 lb. per square inch of cube strength) in excess of 4000 lb. per square inch, as in the example which follows.

EXAMPLE No. 15. Given.—Triangular section. $b = 14$ in. $d = 7$ in. $A_s = 1{\cdot}80$ sq. in. $A_s' = 0$. $f_y = 45{,}000$ lb. per sq. in. $f_{cyl} = 6300$ lb. per sq. in.; therefore $f_u = 0{\cdot}85 \times 6300 = 5350$ lb. per sq. in. To determine.—Ultimate bending resistance of the section.

Solution.—$\bar{p} = \dfrac{1{\cdot}80}{7 \times 14} = 0{\cdot}01836$; $\bar{q}_{y0} = \nu = 0{\cdot}01836 \times \dfrac{45{,}000}{5350} = 0{\cdot}154$.

From Nomogram No. 5 (with $\gamma = 0$, $\nu = 0{\cdot}154$), $\alpha = 0{\cdot}554$, but

$$\varepsilon_s = \frac{45{,}000}{30 \times 10^6} = 0{\cdot}0015;\quad n = \frac{0{\cdot}003}{0{\cdot}0015 + 0{\cdot}003} = 0{\cdot}667.$$

$\alpha = \left[0{\cdot}85 - \left(\dfrac{6300 - 4000}{1000} \times 0{\cdot}05\right)\right] \times 0{\cdot}667 = 0{\cdot}493$; hence the mode of failure of the section is controlled by crushing of the concrete. Thus

$$m_u\bar{p} = \frac{30 \times 10^6 \times 0{\cdot}003}{5350} \times 0{\cdot}01836 = 0{\cdot}309$$

and the compatibility of strains leads to the equation

$$\alpha - \frac{\alpha(2-\alpha)}{2} - \frac{(0{\cdot}74 - \alpha)}{\alpha} \times 0{\cdot}309 = 0, \text{ from which } \alpha = 0{\cdot}517.$$

From Nomogram No. 6 (with $\gamma = 0$, $\alpha = 0{\cdot}517$), $\bar{C}_{ut} = 0{\cdot}0875$.

$$\bar{M}_{ut} = 0{\cdot}0875 \times 5350 \times 14 \times 7^2 = 321{,}000 \text{ in.-lb.}$$

Thus $n = \dfrac{\alpha}{0{\cdot}74} = \dfrac{0{\cdot}517}{0{\cdot}74} = 0{\cdot}698$,

$$\varepsilon_s = \frac{1-n}{n} \times \varepsilon_u = \frac{1 - 0{\cdot}698}{0{\cdot}698} \times 0{\cdot}003 = 0{\cdot}001297$$

and $f_s = E_s\varepsilon_s = 30 \times 10^6 \times 0{\cdot}001297 = 38{,}900$ lb. per sq. in., which is less than the yield point stress $f_y = 45{,}000$ lb. per sq. in.

Mode of Failure.—*Figs.* 9 and 10 (pages 271 and 273 of the number for August 1961) should be transposed. The diagram on page 271 depicts conditions when failure is controlled by crushing of the concrete, and that on page 273 applies when controlled by yielding of the steel.

REFERENCE.

3.—" Ultimate Strength of Non-rectangular Structural Members." By A. H. Mattock and L. B. Kriz. Journal of the American Concrete Institute, January, 1961.

* Concluded from August, 1961.

Book Reviews.

"Failure and Repair of Concrete Structures." By S. Champion. (London: Contractor's Record Ltd. 1961. Price 35*s.*)

It can be said with truth that the author of this book has succeeded in his purpose of compiling a useful report dealing with the engineering rather than the scientific aspect of the deterioration and maintenance of concrete structures. The importance of the correct diagnosis of the cause of a failure is stressed. Many cases of defects due to chemical attack are considered and the part played by the cement in such cases is emphasised. The short early part of the book, which deals with such cases together with mechanical causes of failure (that is overstressing, shrinkage, thermal and moisture changes, and the like), is followed by the major part which deals in a practical manner with methods of repair. The scope of this part is extensive since it ranges from dealing with cracks (a particularly good section) to underpinning. The special case of failure of prestressed concrete structures is considered briefly but realistically and fifteen potential causes of failure are itemised.

"The Elementary Principles of Reinforced Concrete Design." By W. H. Elgar. (London: The Architectural Press. 1961. Price 18*s.* 6*d.*)

As the title implies, this book gives the bare outlines of the basis of design of simple reinforced concrete building components, and as such is a suitable introduction to the subject for students of architecture and building for whom it is intended. Even so, the load-factor method, flat slabs, shells and prestressed concrete are included but are necessarily dealt with in a superficial manner. Although there are only about a hundred pages of text in large type with numerous exceedingly clear diagrams, the contents include most of the salient matters expected to be covered in an elementary book. Many examples are given, although some of the few details of reinforcement may be open to criticism. The introduction contains a number of weak statements, which may be misunderstood by a thoughtful reader, but elsewhere the phraseology is good and concise.

"Innovations in Building Materials." By Marian Bowley. (London: Chatto & Windus Ltd. 1960. Price 70*s.*)

This book is one of a series on industrial innovations. The authoress is a reader in political economy and the work comprises a considerable collation of documents, data, and descriptions of various aspects of the building industry, but in particular the materials brick, concrete and glass, with some reference to aluminium and asbestos-cement. The term innovations is used fairly loosely since the classification relates to developments since the early nineteenth century, and to innovations before and since the first world war.

The subject-matter is drawn from numerous sources, but nevertheless the scope of publications searched for information seems to be limited especially where concrete is concerned. The references for this material are largely to architectural journals. There are, however, some interesting commentaries on the cement and precast concrete industries.

Since the writing, although factual, is somewhat discursive, it would be an advantage if the conclusions given at the end of each chapter were more concisely written instead of being equally wordy and containing many diversions.

"Architects' Detail Sheets." Edited by E. D. Mills. (London: Iliffe Technical Publications Ltd. 1961. Price 35*s.*)

It is generally believed that most reinforced concrete detailers lack knowledge of practical building construction. (It is likewise alleged that many architectural draughtsmen are deficient in practical knowledge of structural engineering details.) If the former statement be true, these sheets of sketches, which are the fifth in a series of similar publications, will help greatly to meet the deficiency. As in the previous publications, ninety-six large and clear drawings are given of various building details, including the attachment of various building components to concrete floors, roofs, balconies, walls and stairs. These are not merely text-book details, since most are taken from actual named structures. Although many of the details relate to concrete, others are concerned with construction in brickwork, timber and steel.

Concrete Avalanche Sheds.

Fig. 1.

THE accompanying illustrations (*Figs.* 1 and 2) show a concrete tunnel 1036 ft. long in the course of construction on the Trans-Canada Highway. The tunnel, which is of precast and cast-insitu reinforced concrete, is one of a number required to protect the road from avalanches so as to maintain an all-weather route through the Rocky Mountains. It is situated 25 miles to the east of Revelstoke, British Columbia. It was constructed in 3½ months in time for the winter of 1960. Instructions were given to the workmen that the site was to be vacated immediately in the event of warning of a 65-per cent. probability of an avalanche occurring. The necessity for completion by the middle of November is emphasised by a warning of a 55-per cent. probability which occurred on November 3, 1960.

All plant, equipment and stores had to be transported 450 miles by rail from Vancouver and, because of the urgency of the work, over two thousand concrete units were precast in Vancouver. For the cast-insitu work nearly 1000 tons of reinforcing steel was delivered to the site. A labour camp to accommodate up to 150 men was erected near the site.

The tunnel is designed to withstand a vertical load of 1100 lb. per square foot. The structure has a clear internal span of 36 ft. and an internal height of 14 ft. 6 in. at the front and 21 ft. at the rear. Precast concrete columns 12 in. by 30 in. in cross-section are provided at 7-ft. centres

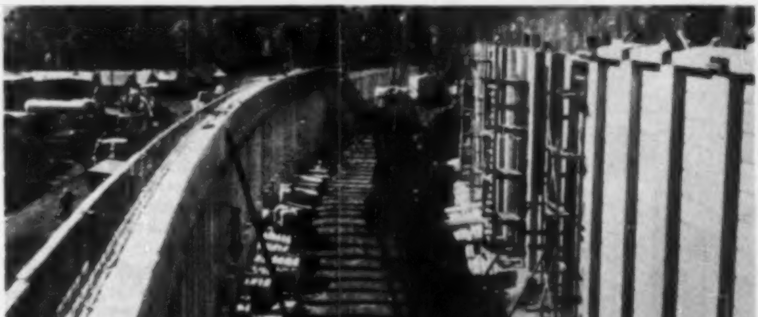


Fig. 2.

at the front, and cast-insitu columns 48 in. square are provided at the rear at 14-ft. centres. The rear columns are connected by tie-beams 40 ft. long to a continuous anchor-wall 2 ft. wide and 8 ft. deep. The roof comprises prestressed precast beams, each about 40 ft. long, 42 in. wide and 36 in. deep, laid alongside each other.

The consulting engineers acting for the Government of British Columbia are Messrs. Choukalos, Woodburn, Hooley and McKenzie. The contractors were Messrs. John Laing and Son (Canada), Ltd., which firm has recently been awarded contracts for the construction of two further avalanche tunnels over the Trans-Canada Highway near Lanark, also in British Columbia. The new work commenced recently and must be completed by early November this year to avoid danger from avalanches. The construction is to be similar to the earlier work and comprises cast-insitu and precast reinforced concrete. More than 1700 precast units are to be used, including about three hundred 17-ton prestressed precast roof beams. Over 15,000 cu. yd. of excavation, much of it in rock, will be necessary and about 600 tons of reinforcing steel are to be used.

Bags for Batching Concrete Materials.

A NEW type of container for transporting or storing the materials, including the water, for a batch of concrete in an unmixed condition and in the proportions required, has been developed recently in the U.S.A. by Rodeffer Industries, Inc., of California. The container is a collapsible rubber bag 6 ft. high and divided into two compartments having a combined capacity of 1½ cu. yd. The smaller central cylindrical compartment contains the cement, and is sealed off from the outer annular compartment which contains the coarse and fine aggregates and the water. While being filled the empty bag is hoisted on to a movable platform below the outlets of a batching plant from which all the ingredients for the concrete are discharged into the bag. The bag is then transferred to a 5-ton fork-lift truck which conveys it to a store, or, by other means, the filled bag is transported to the site. When concrete is required, the bag is hoisted bottom upwards (*Fig.* 1), a pin is withdrawn and the contents are discharged directly into the mixer.

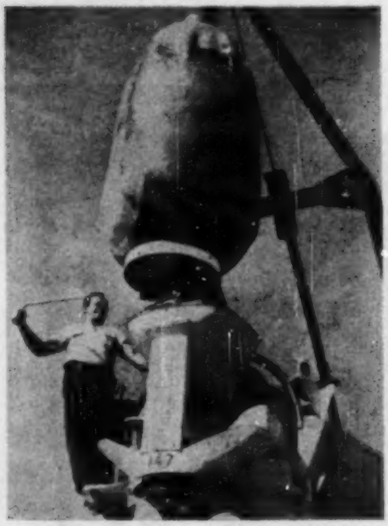


Fig. 1.

It is claimed that closer control of the concrete mixture is obtained and that few alterations are required to be made to a batching plant at which the bags are filled. It is also claimed that a bag can be used up to a thousand times before replacement is necessary, and when empty it can be dropped safely to the ground if mixing concrete at a height. It is estimated that in the U.S.A. the use of large numbers of these bags may reduce the cost of batching concrete by as much as 75 per cent. Fluctuation of demand for concrete at the site can be dealt with as filled bags can be held in stock until required, thereby enabling batching plants to be in continuous use irrespective of the immediate requirements.

Standard Prestressed Beams.

IN the number of this journal for May 1961, dimensions are given of standard prestressed concrete beams in accordance with a report recently issued by the Prestressed Concrete Development Group. The first bridge incorporating such beams has been built by the Central Electricity Generating Board and provides access over a main drainage channel to a substation at Whitson, near Newport, Monmouthshire. The bridge has a clear span of 46 ft. and allows for future widening of the channel. The design load was based on the heaviest load which it is anticipated will cross the bridge, namely a 165-ton transformer; beams of one of the standard sections and 48 ft. long, with a cast-insitu reinforced concrete deck, are capable of carrying this load. The reinforced concrete abutments are supported on piles 40 ft. long. The bearings for the beams (*Fig.* 2) are bull-head rails laid on each abutment, on which bears a flat steel plate embedded in the underside of each beam.

Since it was impracticable to launch the beams into position from the side, as a public road would thereby be obstructed, a temporary bridge was built on to which the low-loader delivering the beams was driven. The beams were lifted off the low-loader and placed in position by two cranes, one at each abutment, working together (*Fig.* 1). The unloading and placing of each beam took about five minutes; all the beams were placed in two days.

The bridge was designed by the Transmission Project Group of the Central Electricity Generating Board. The contractors were Messrs. A. Jackaman & Son, Ltd. The precast beams were made by Anglian Building Products, Ltd.

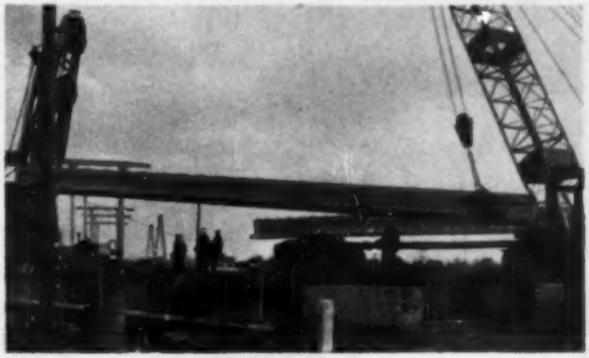


Fig. 1.

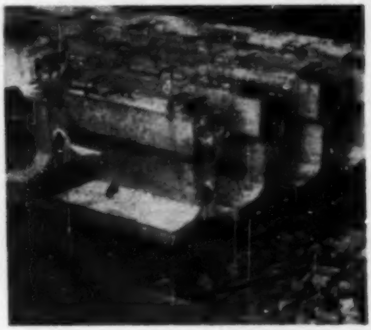


Fig. 2.

Change of Address.

THE Indented Bar and Concrete Engineering Co., Ltd., have now removed to 67-69 Salcott Road, London, S.W.11 (Telephone—BATtersea 4121).

British Standard Notation for Concrete Structures.

PART 4 of British Standard No. 1991, "Recommendations for Letter Symbols, Signs and Abbreviations", was published recently and deals with the notation for "Structures, Materials and Soil Mechanics". In general the symbols for reinforced concrete conform to those in B.S. Code of Practice No. 114, but many more items are given.

The terms in the following are exactly as given in the Standard, which also includes basic symbols, and symbols for use in steel structures, aircraft structures, soil mechanics and stress analysis. Standard abbreviations for units of measurement and the like are also given. The complete list for concrete structures is as follows.

DIMENSIONS

Length or span	l, L
Height	h
Depth	d, D
Effective depth from compressive face to tensile reinforcement	d_1
Depth from compressive face to neutral axis	n_1d_1, d_n
Depth from compressive face to compressive reinforcement	d_2
Breadth	b
Thickness	t
Radius	R
Diameter	D
Lever arm	a_1d_1, l_a, a
Eccentricity	e
Pitch	s
Sum of perimeters of tensile reinforcement	o
Area of column or beam	A
Area of concrete	A_c
Area of steel	A_s
in tension	A_{st}
in compression	A_{sc}
Area of web or shear reinforcement	A_v
Equivalent area of helical or binding reinforcement (volume of helix/length of column)	A_b
Area of concrete in column core	A_k

PROPERTIES OF SECTIONS AND MATERIAL CONSTANTS.

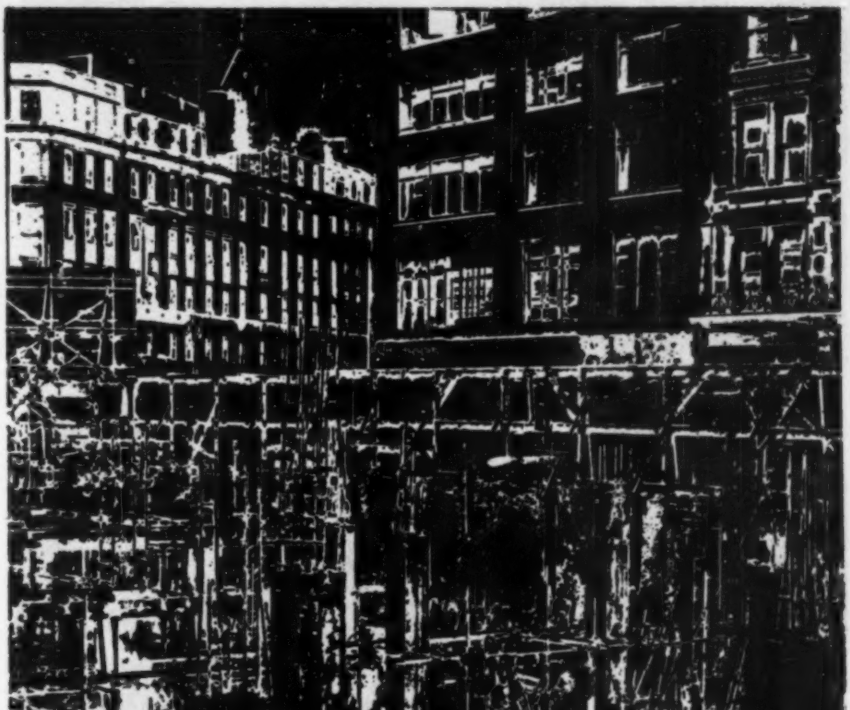
Moment of inertia; second moment of area	I
Radius of gyration	k
Modulus of section	Z
Moment of resistance in bending	M_r
Flexural stiffness of a member (I/l)	K
of a beam	K_b
of an upper column	K_u
of a lower column	K_l
Percentage of reinforcement	r
Equivalent area of section in terms of concrete	A_e
Modulus of elasticity (direct) or Young's modulus	E
of concrete	E_c
of steel	E_s
Modular ratio (E_s/E_c)	m
Modulus of rigidity, or shear modulus	G
Bulk modulus (modulus of compressibility)	K
Poisson's ratio	ν . . . σ

EXTERNAL LOADS AND PRESSURES.

Load or weight, concentrated or total	W
Load or weight, distributed (divided by length or area)	w
Vertical reaction	V
Horizontal reaction	H
Resultant reaction	R

FORCES AND MOMENTS.

Direct force	F
in concrete	F_c
in compression	F_{cc}
in tension	F_{ct}
in steel	F_s
in compression	F_{sc}
in tension	F_{st}
Permissible force	P
in concrete	P_c
in steel	P_s
Shear force	Q
Bending moment	M
Fixed end moment	M_e



STRESSES.	
Direct stress	f
in concrete	f_c
in compression	f_{cc}
in tension	f_{ct}
in steel	f_s
in compression	f_{sc}
in tension	f_{st}
Permissible stress	p
in concrete	p_c
in compression	p_{cc}
in compression in bending	p_{cb}
in steel	p_s
in compression	p_{sc}
in tension	p_{st}
Concrete strength	
Cube strength	u
Equivalent cube strength	u_e
Cylinder strength	u_{cyl}
Preliminary cube strength	u_p
Works cube strength	u_w
Ultimate compressive strength in bending	u_b
Modulus of rupture	R
Steel strength	
Ultimate stress	f_u
Proof stress	f_p
Yield stress	f_y
Shear stress	q
STRAINS AND DISPLACEMENTS.	
Direct strain	ϵ
in concrete	ϵ_c
in steel	ϵ_s
(with further modifications of subscripts as for direct stress if required)	
Shear strain	ϕ
Deflexion	δ, Δ
Angle of rotation	θ

A Large Prestressed Concrete Transmission-line Tower.

In this journal for June last, a brief description is given of a prestressed concrete transmission-line tower almost 100 ft. high, in Massachusetts. It has been pointed out by Messrs. Schupack and Zollman, the consulting engineers responsible for the design of the structure, that the wire guys described as stabilising the tower were attached only during erection but they were never used effectively. The tower is entirely a free-standing structure cantilevering vertically from the pile-caps.

Bulletins Received.

"Annual Report of the City of Manchester Surveyor and Engineer's Department for 1960."—Contains illustrations and descriptions of civil engineering works carried out in 1960.

"Better Opportunities in Technical Education." Ministry of Education. (London: H.M.S.O. 1961. Price 1*s.* 3*d.*)—Gives the Government's proposals for reorganising the existing system of the technical college courses for technicians, craftsmen, and operatives.

"Rectangular Flat Slabs Supported on Columns." By H. Nylander. Bulletin No. 34. The Royal Institute of Technology, Stockholm. (In Swedish with a summary in English. 1959. No price stated.)

"Splicing of Cold-stretched Reinforcing Bars with Anchoring Rings." By C. Forssell. The Royal Institute of Technology, Stockholm. (In Swedish with a summary in English. 1960. No price stated.)

"Punching of Concrete Slabs Without Shear Reinforcement." By S. Kinnunen and H. Nylander. Publication No. 158. The Royal Institute of Technology, Stockholm. (1960. In English, Price Kr. 11.)

"The Temperatures Under Houses Erected Immediately on the Ground and the Heat Losses from their Foundation Slab." By O. Vuorelainen. Publication No. 55. The State Institute for Technical Research, Finland. (1960. In English. No price stated.)

"Suggestions for Requirements of Sound Insulation." The State Institute for Technical Research, Finland. (1960. In Finnish with a summary in English. No price stated.)

Detailing Reinforced Concrete.

The City and Guilds Institute of London announce that they are to hold a course on the detailing of reinforced concrete structures. This course should enable those engaged in such work to attain a higher standard than is generally prevalent. This journal joins with the Joint Committee on Structural Concrete in hoping that design engineers will encourage their assistants to attend the courses which will eventually be held at various technical colleges throughout the country.

Computer Programmes for Structural Problems.

THE computer programmes prepared by the Cement and Concrete Association are given in the following list. Most of the programmes relate to structural analysis and are applicable to any material. Those that are suitable for prestressed concrete are marked with an asterisk. The Association should be consulted regarding the availability of these programmes and the type of computer for which each has been devised.

Solution of simultaneous linear equations.

A single-plane structural frame.*

Stressing of multiple-anchor 3-dimensional pipework systems.

Linear pin-jointed frames.*

Fundamental properties of beams of varying section.*

Composite action between beams and slab under transverse load.*

Elastic analysis of rigid-jointed plane structures.*

Stressing and deflections in piping systems.

Surveying by quadrilaterals.

Calculations of sag and tension of suspended cables.

Single-grid frames.*

Solution of a large number of simultaneous linear algebraical equations.

Solution of second-order partial differential equations with particular reference to torsion problems.

Simultaneous linear ordinary differential equations.

Cut and fill. (Two programmes.)

Plane frames, grillages, and space frames.*

Elastic skew slabs.*

Prestressed concrete sections *

Composite action between beams and slabs.*—I. Slab with identical span-wise beams repeated at equal intervals supported over several transverse supports; and with beams whose flexural stiffness varies across the span. (Two programmes.) II. Rectangular slab resting on four edge-beams and supported at the corners on rigid stanchions. (Two programmes.) III. Rectangular stiffened slab on elastic foundation.

Moment distribution.*

Laterally loaded piles.*

Slip circle analysis of the stability of earth dams.

Continuous beams.*—I. Slope deflection calculations. II. Fixed-end moments. III. Quarter-point deflections.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", September, 1911.

Agricultural Products.

THE accompanying illustration, which shows one of the stands at the Royal Agricultural Society's Show held in Norwich in the summer of 1911, is of interest in view of the fact that concrete products and buildings are to be a feature (as they were last year) at the Royal Dairy Show to be held at Olympia, London, next month. The products exhibition in 1911 included cattle and pig troughs, flooring and roofing tiles, fence posts, cattle pens, rick posts, and blocks for walls.

The Late Mr. G. A. Maunsell.

MR. GUY ANSON MAUNSELL, whose death at the age of 76 occurred in June last, was well known for his interest in concrete and will be remembered for his designs, many of which were in advance of the practice of their day. His early civil engineering training was in connection with hydro-electric works in Switzerland and the naval base at Rosyth. He served in France in the Royal Engineers in World War I, and early in 1918 he was working on the design and construction of concrete ships and " mystery " towers. In World War II, he was responsible for the design of floating thin-slab concrete structures (see this journal for April, 1946).

Between the wars, he was first at the Ministry of Transport, then on the Falls of Clyde hydro-electric works with Sir William Arrol & Co., Ltd., and later on the widening of Putney Bridge with Messrs. Dorman Long & Co., Ltd. He then became managing-director of the Anglo-Danish Construction Co., which was formed for the construction of the famed Störstrom Bridge. In 1936 he opened a consulting engineering practice, in Westminster, from which he retired in 1959, and in which he was responsible for many well-known major structures.

Mr. Maunsell was a forthright all-round civil engineer; there are those who may remember that, in the mid-twenties, he might proceed with the construction of, say, a foundation in accordance with a sound practical design of his own devising without awaiting the delayed arrival of more intricate details from the drawing office.

Technical Writing.

A COURSE of six lectures, commencing on October 20, 1961, on " The Technique of Technical Writing " are to be given by various lecturers at Borough Polytechnic, London. The subjects dealt with will include style, common errors in style, sources of information, technical illustration, printing, and publishing. The lectures, particulars of which can be obtained from the Secretary, Borough Polytechnic, Borough Road, London, S.E.1, are intended for graduates (or equivalent status) with some experience in technical writing.

The Largest Dry Dock in the World.

CONCRETING in connection with the dry-dock at the Puget Sound Naval Shipyard, Washington, U.S.A., is now proceeding. The dock, which will be the largest dry dock in the world, is 1152 ft. long and 180 ft. wide, and there will be 42 ft. of water at mean low water.

The first stage of dredging unsuitable material, backfilling with selected material, cofferdam construction and de-watering has been completed. This work necessitated the removal of 600,000 cu. yd. of material. Below mean low-water level the material was removed by a suction dredge and by three derricks, with 3-cu. yd. buckets, mounted on barges. Selected filling was brought to the site and pumps mounted on barges sluiced the material into the water. Backfilling was built up along both sides of the dock to a level about 40 ft. below mean low-water level; the central portion was left at about the level of the base of the dock. A row of steel sheet-piles was driven along each side of the dry dock to form part of a cofferdam and will remain in place as part of the permanent construction. The outer end of the cofferdam consists of eleven circular sheet-pile cells of 60 ft. diameter. About 600,000 cu. yd. of backfilling was placed to complete the walls of the cofferdam. Before the cofferdam was closed sixty-six deep wells were sunk around the perimeter of the site, about 35 ft. inside the sheet-pile walls, for dewatering purposes. Three-hundred-and-ninety well-points were then installed at 10-ft. centres within the wells.

The second stage is the casting of the concrete floor and walls of the dock. Most of the floor will be a 7-ft. slab of reinforced concrete which increases to a thickness of 12 ft. under the main walls. The slab is being cast in sections 24 ft. by 40 ft. within steel shutters. The side walls, which are of reinforced concrete 12 ft. thick at the bottom and tapering to 2 ft. 9 in. at the top, are being cast in two stages, also using steel shutters. The walls have buttresses of precast concrete supporting galleries, tunnels and platforms at the top.

Patent Applications for Shuttering.

Shuttering is supported from tubular scaffolding (*Fig.* 1) by a turnbuckle comprising two rods 10, 11 with threads of opposite hand adjustably connected to a correspondingly threaded connecting loop 12 and each having a rotatable cross tube 15, 20 at its outer end. The tube 15 is held against axial movement within a collar 13 by welded abutments 16. The tube 20 is similarly held within two sleeves 19. In use, the tube 15 is laid on two horizontal scaffold tubes 21 and the shuttering supported from a tube 22 clamped at 23 to the cross tube 20. The turnbuckle may also be used horizontally. Specification 852,830 is referred to.—No. 852,918. May 16, 1958.

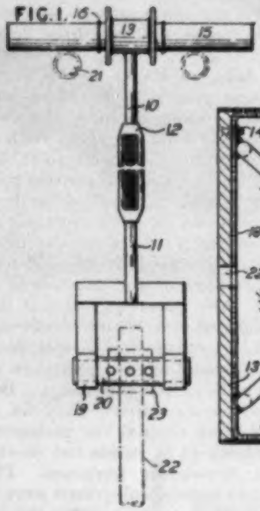


FIG. 1.

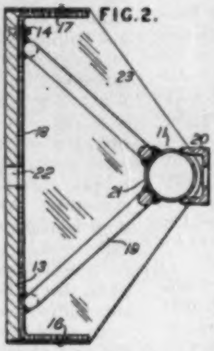


FIG. 2.

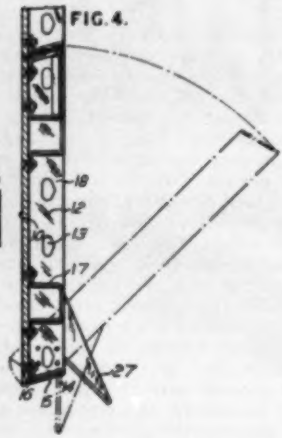


FIG. 4.

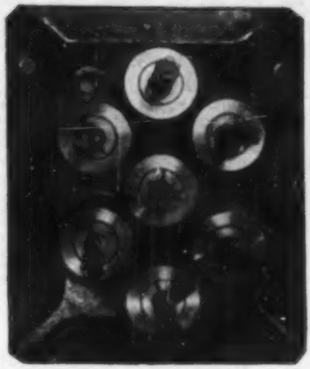
The soldier for concrete shuttering (*Fig.* 2) of the parent Specification has a concrete engaging face comprising a sheet 13 of resin bonded plywood screwed at 14 to angle bars 18 and surrounded by side sheets 17 of metal and a metal end plate 23. Zig-zag bars 19 are welded to the bars 18 and a bracing tube 11. In use, tie rods are passed through holes 22 and 21 to be tightened against short lengths of channel 20. The soldier is attached to adjacent shuttering by holes 16.—No. 853,591. October 28, 1958.

In the shuttering for casting concrete (*Fig.* 4) of the parent Specification comprising panels having flanges 12 connected by bolts in slots 13 parallel with the face 10 of the panel, the edges of slots being inclined so that by forcing each bolt to a narrow end of a slot where the bolt engages both edges the edges and thus the faces 10 of the shutters are aligned, the facing member 10 is non-metallic, e.g. plywood. The flanges may be of angle section bar 14 to which is welded a strip 15 extending laterally beyond the bar 14 so as to protect the edge of the facing member 10, which is attached by bolts 16 to the bar 14 and cross-members 17 welded to the flanges 12. The panel 18 shown in Fig. 4 is adapted to be swung about bolts in the bottom slots 13 so that concrete can be poured into the shuttering. Stops 27 limit the opening of this panel.—No. 853,278. December 16, 1958.

[The foregoing Applications are in the name of Kwikform Ltd.]

Data for Pricing Reinforced Concrete.

Current prices are as given on page 296 of this journal for August, 1961.



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SITUATION VACANT. Engineering assistant required to work in structural engineering department. Applicants should have minimum of three years' experience in reinforced concrete or steelwork. Permanent position. Five-days' week. Luncheon vouchers. Apply General Manager, Atcost (Factories) Ltd., York House, High Street, Tunbridge Wells.

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SITUATIONS VACANT. Draughtsmen required, both junior and senior, to do work on precast or insitu, prestressed or reinforced concrete work. Salaries up to £1,100 per annum will be paid. Apply to the Chief Engineer, The London Ferro-Concrete Co. Ltd., Maxted Close, Hemel Hempstead, Herts.

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Conditions of appointment and information as to the method of application may be obtained from THE SECRETARY, ASSOCIATION OF UNIVERSITIES OF THE BRITISH COMMONWEALTH (Branch Office), Marlborough House, Pall Mall, London, S.W.1.

Applications close on 20th October, 1961.

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Published by Concrete Publications Ltd., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
Printed by Butler & Tanner Ltd., The Selwood Printing Works, Frome and London